AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

319.

(Vol. XV.—February, 1886.)

THE DISINTEGRATION OF THE EGYPTIAN OBELISK IN THE CENTRAL PARK, NEW YORK.

By THOMAS EGLESTON, PH.D., M. Am. Soc. C. E.

READ JANUARY 20TH, 1886.

The decay of the stone of the Obelisk in the Central Park of New York has attracted a great deal of attention, on account of the publicity which has been given to its having been coated with paraffine in order to preserve it. I have therefore thought it would be of interest to this Society to know of the results of a special examination which I have made of it, in addition to those already communicated.* It was supposed when the Obelisk was brought here that it was in a perfect condition, and that it had not suffered at all by its exposure to the climate of Egypt for 3 500 years. A careful examination of the Obelisk itself, and of the pieces which have been detached from it, shows, however, that this was not the case, but that disintegration has been going on for a very long time in the interior of the stone, which has only become apparent within a few months, owing to the fact that, until its erection in the Central Park, there were no causes which

* Decay and Preservation of Building Stone. By T. Egleston, Ph.D. Annual Convention Am. Soc. Civil Engineers, June, 1885. To be published hereafter.

tended to attract attention to the weakness of the stone. For some months it has been noticed that pieces of the Obelisk have been falling about the base. These pieces, mostly of small size, attracted the attention of those interested in its preservation, and led Mr. A. Hague, during the last summer, to make an investigation of the condition of the whole shaft while the scaffolding used by the workmen in putting on the paraffine coating was still up and they were chipping off the cracked pieces and heating the surface of the obelisk preparatory to putting on the paraffine. This examination showed that the south and west faces, which were exposed to the direct rays of the sun, were the most weathered, but that it was cracked on all sides, showing that a process of decay had begun which is likely to lead to most serious consequences if the Obelisk is not protected.

The process of decay through which the stone is passing is not of the chemical nature which has been attributed to it. This is shown not only by an inspection of the Obelisk itself, but by the microscopic examination of the pieces of the stone detached from it. The first thing done by the workmen in getting the stone ready for the coating which was to protect it, was to pick off all the pieces which were already so much cracked as to be easily separated, with small iron tools. In doing this, pieces of an inch square up to those which were over a foot long and nearly two inches in thickness were detached. These pieces appear to the unaided eye to be of perfectly natural stone. Many of them are coated with a green substance which I at first sight supposed to be a mineral, and which, from its peculiar color, I took to be epidote. A careful microscopic examination, however, did not show any crystals or fibers, which made me withdraw the determination, and decide that it must be some kind of vegetable stain. Mr. P. H. Dudley, who has made a careful microscopic examination of this stain, pronounces it to be a very low order of vegetable growth belonging to the genus *Protococcus*, probably the species *pluvialis*, a variety not found in this country, and known mostly in Egypt. What strikes one at first, in making an examination of the pieces of stone, is their remarkably fresh appearance, except where they are colored green. The crystals of feldspar are large and rosy, and do not, to the eye, show any sign or trace of decomposition. Where the fracture of the stone is quite fresh it looks like any other syenitic granite, with a considerable preponderance of hornblende, and but a small quantity of mica. To the naked eye, only quartz, feldspar, hornblende and mica

are visible. Under the microscope, tourmaline, zircon, titanite and magnetite are seen in small quantities with perfectly fresh fractures, and not showing the least sign of decomposition. With a 1½-inch objective and powerful light, I first began to notice small cavities, here and there, of irregular shape, and quite deep, sometimes entirely empty, but often containing a substance having very much the appearance of the hornblende and pyroxene in specimens which occur in limestone, and have been placed in acid to detach the crystals. These cavities, though not numerous, occur only in the vicinity of the hornblende. On searching for them I found them relatively numerous when all the surfaces of attack were taken into consideration, and sometimes so deep that their bottom could not be seen with an inch objective. Pittings, easily distinguishable after a little study, giving the quartz and the feldspar the appearance of having been etched very slightly with hydrofluoric acid, were quite visible. They were not uniform, but were sufficiently numerous in places to give the surface a greasy appearance. In some places the crystals of hornblende were completely gone, leaving the shape of their crystals in the rock. I did not notice any decay in any of the other minerals, and this one seemed to be most prominent in the vicinity of the vegetable growth, but I was unable to determine whether this appearance of a part of the quartz and feldspar, having gone into solution, was owing wholly or in part to it.

The quantity of the pieces removed from the Obelisk previous to putting on the paraffine is reported to have been as much as 800 pounds, and was probably considerably more than that. These pieces have been freely circulated, and can be frequently seen in the possession of curiosity hunters in different parts of the United States, some of them having a few pieces and others several pounds. What the future of the monument is to be is a matter of some anxiety. It is certain now that the opinion advanced at the time that the Obelisk was erected in Central Park, that the stone was quite sound when it was erected there, was a mistake. It was full of minute cracks when it came here. This is shown by the rapid disintegration of the stone, which has suffered more in the five years of exposure in the Central Park than in the 3 500 of its previous history, and also by the fact that the cracks are filled with vegetable life which received sufficient light through the stone to be colored yellow and green by the chlorophyl, the depth of the stone being shown by the variation of tinge of green. In

moist places this kind of vegetable growth does not take long to propagate itself, but in dry ones, like the interior of the cracks in granite, it must have taken many years. To have affected the stone even slightly, as it appears to have done, must have taken many more. It is quite possible that the germs of the vegetable matter were in the stone when it was in Egypt, and did not develop there because they did not have sufficient moisture to cause them to germinate, and that they only commenced to grow when brought here. It is also quite possible that this vegetable growth is not the cause of any of the apparent solution or pitting, though the appearances are exactly like those of other minerals whose surfaces have been slightly acted on; but it is certain that, while the chemical action on the stone is very slight, there has been a deep-seated and prolonged disintegration of the rock from purely physical causes, producing cracks and ruptures along the lines of greatest weakness, and that the stone has no longer the strength to resist the atmospheric influences of our very changeable climate, where the extremes of heat and cold are so very great, and where, as compared to Egypt, the amount of moisture in the air is always at a maximum.

The method of applying the present protecting (?) coating seems to have been a fatal mistake. Nothing of any account has been dissolved out of the stone, there is, therefore nothing to be replaced. If there had been, paraffine in solution would have been one of the best materials to fill them. Granite is not porous, there were, therefore no cavities to be filled. The stone being full of cracks from natural causes, the heat which was used to cause the paraffine to sink into the body of the stone, when applied to the outside would cause an expansion, which would not be responded to by the interior of the granite, and the cracks already there would increase in size, and pieces would chip off as they did, and new cracks would be formed in the stone, already weakened by long exposure. This is distinctly traceable on almost every specimen chipped off, the old part of the crack showing the stone somewhat less fresh on the surface, while the part recently cracked is quite fresh.

It is expected, now that the Obelisk is supposed to have been waterproofed, that the disintegration will cease, but this appears to me to be founded on an altogether mistaken theory, which is, that the cracking is alone due to the expansion of the ice formed in the cracks. The rapid and extreme changes of temperature in this climate in a stone, which,

from its mass alone, must have but a feeble conducting power, would be sufficient to cause the disintegration already begun, in a stone weakened by exposure to great heat in a dry climate, to continue with comparative rapidity without the intervention of ice, but simply from the continued expansion and contraction going on on its weakened surface. But in a moist climate like our own, where it was subjected to both extremes of heat and cold, it would take place rapidly, as it has done.

As it was a matter of interest to ascertain how far moisture had to do with the cracking under heat, I made the experiment of submitting pieces of granite which had become quite dry from having been kept housed many years, to as high a heat as could be obtained in the laboratory without melting, and to my surprise found that no spalling or even cracking occurred, although the pieces were subjected to the heat suddenly and for varying periods of time. It is well known that granite in buildings, when subjected to fire, spalls. This is owing to the moisture it contains; to the expansion of gas and liquids contained in microscopic bubbles in the quartz; and to the want of conductivity of the stone itself. Perfectly dry granite does not spall unless exposed very suddenly to a very high temperature. No granite, however, exposed to the weather in this climate is ever dry. Fresh granite contains about one per cent. of moisture. That weakened by age, like the surface of all the obelisks, may contain many times that amount, consequently all granites on the outside of structures do spall when exposed to fire. From the fact that the stone of the Central Park Obelisk is already weakened and probably full of fissures, which in this climate will tend to develop year by year, and from the very fact that the disintegrated stone will absorb more moisture than stone which is fresh, it seems probable that no protection or coating given to the stone will arrest the process of disintegration already commenced in it, if it is left exposed. Even if the surface was entirely waterproofed, the cold of winter and the heat of summer would act below the surface both of the coating and of the stone, causing the coating to break or fissures through it to occur, so as to let in the moisture, and then both causes would operate together as before. But in any case heat and cold will act altogether independently of moisture, whether the outside be coated or not, and further disintegration must take place under the same circumstances and conditions as that which has already so much weakened the stone. Placing the Obelisk in the Central Park, where it

is exposed to nearly every agency that could tend to destroy it; allowing the surface of a stone already so much weakened by disintegration to be heated, thus causing further cracks to be made in it; is a greater monument to public indifference and ignorance than the shaft ever was to the dignitary who first erected it, or the events chronicled in its hieroglyphics.

The same dangers, if the reports about it are true, threaten the Obelisk on the Thames Embankment, although not to the same extent. The climate of England is much less severe than ours, both in winter and in summer. The causes for disintegration being the same in both cases, will affect the London Obelisk less than ours, and there seems to be, so far as any examination of it has been made, no great present danger in leaving it exposed where it is. The Obelisk in the Place de la Concord, in Paris, is reported cracked all over its surface. Both the European Obelisks are therefore in danger of being seriously damaged within the next hundred years. Housing seems to be the only thing left for the Obelisk in Central Park.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

320.

Vol. XV.—February, 1886.

FORMULAS FOR THE WEIGHTS OF IRON AND STEEL RAILWAY BRIDGES UNDER STANDARD SPECIFICATIONS.

By GEORGE H. PEGRAM, M. Am. Soc. C. E.

PRESENTED AT THE ANNUAL CONVENTION, JUNE 10TH, 1884.

WITH DISCUSSION.

It has been asserted that there is no law governing the variation of the weights of iron and steel bridges, and that, therefore, any formulas purporting to give them with exactness, or even approximate exactness, should be viewed with suspicion.

In the writer's opinion, this feeling at the present day is analogous to the opposition that is made to the use of steel for structural purposes, which does not always involve a proper consideration of the rapid progress which is being made in engineering.

The specifications for bridges and the details of construction are now comparatively uniform, and as the individual eccentricities of engineers become confined within narrower limits, the operation of a general law is proportionately extended.

The following specifications have been assumed. They might be called standard for this country, in the sense that they are the most usual.

Those affecting steel will doubtless be modified as our knowledge of that metal is increased.

Specifications.

The structure shall be proportioned to sustain its own weight, in addition to the moving load as hereinafter given for its particular class.

In order to provide for the effects of impact, the following percentage shall be added to the strains produced by the moving load considered as static:

For spans under 25 feet, 30 per cent.
" " of 80 " 8 "

The added percentage to vary between these limits, in proportion to the length of span:

End suspenders,	25 per cent.
Middle ties,	15 "
First counter-ties,	25 "
Second "	35 "
Last "	50 "

In plate girders the flange stress shall be considered as borne by the flange without the assistance of the web, and the shearing stress as borne by the web alone.

Under these conditions of loading, the stresses shall be limited as follows, viz.:

Wrought-Iron—

10 000 pounds per square inch in tension.
7 500 " " " " " shearing.
8 000 " " " " " compression,

reduced in columns according to the formula $\frac{8\,000}{1+\frac{l^2}{a\,r^2}}$ in which

l = the length in inches.
r = the least radius of gyration in inches.
a = 18 000 for two pin ends.
a = 24 000 " one " "
a = 36 000 " two flat "

Lateral bracing shall be provided to resist a wind pressure of 30 pounds per square foot on twice the surface of one truss, together with a moving train surface averaging ten square feet per foot of span.

The stress in lateral rods shall be limited to 15 000 pounds per square inch, after adding 10 000 pounds per rod for adjustment.

Lateral struts shall be proportioned by the above column formula, with 10 000 in place of 8 000 as a basis.

STEEL—

Lateral rods,	20 000	pounds	per	square	inch.	
All other parts,	14 000	"	"	"	"	in tension.
	14 000	"	"	"	"	" compression,

except in members over sixteen diameters in length, where the column formula for wrought iron, with fifty per cent. added, shall be used.

With some engineers it is customary to vary the unit stresses instead of adding percentages. The result is practically the same, but the system of percentages seems to be more logical, and admits of varying stresses, closely approximating the effects of impact, which it would otherwise be difficult to express.

In the investigations of which this paper is the result, four classes of loading have been assumed, which will be denoted, for convenience, by the initial letter of the engine used.

They are:

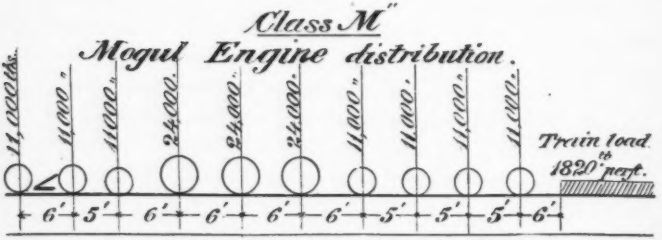
Class M.—Two coupled Mogul engines, followed by a train weighing 1 820 pounds per foot of track, which was the common loading eight years ago, and is now used on some roads.

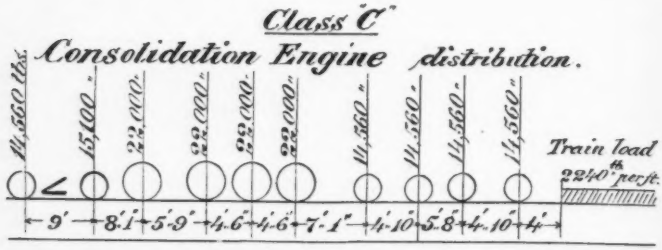
Class C.—Two consolidation engines, followed by a train weighing 2 240 pounds per foot of track, which has been the usual loading for the past five years.

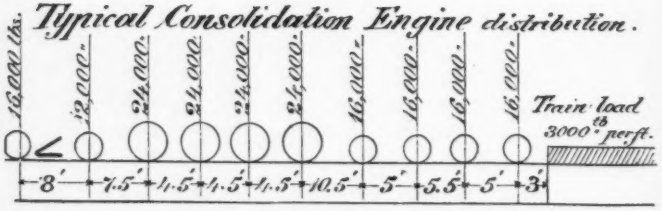
Class T.—Two "Typical" consolidation engines, followed by a train weighing 3 000 pounds per foot of track, which is now coming into use, and will probably be the loading for some time to come.

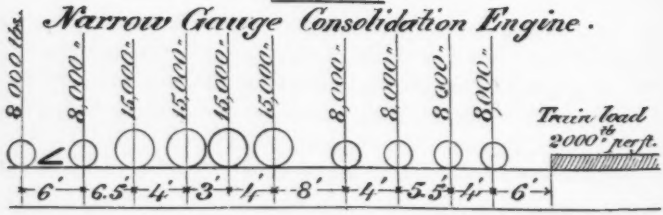
Class N.—Two narrow-gauge consolidation engines, followed by a train of 2 000 pounds per foot of track.

The distribution of weight in the several classes is shown in the following diagrams.


Class M
Mogul Engine distribution.
11,000 lbs.
11,000 "
11,000 "
24,000 "
25,000 "
24,000 "
11,000 "
11,000 "
11,000 "
11,000 "
Train load 1820 lbs per ft.
6' 5' 6' 6' 6' 6' 5' 5' 5' 6'


Class C
Consolidation Engine distribution.
14,560 lbs.
15,100 "
22,000 "
22,000 "
22,000 "
22,000 "
14,560 "
14,560 "
14,560 "
14,560 "
Train load 2240 lbs per ft.
9' 8'1" 5'9" 4'6" 4'6" 7'1" 4'10" 5'8" 4'10" 4'


Class T
Typical Consolidation Engine distribution.
16,000 lbs.
12,000 "
24,000 "
24,000 "
24,000 "
24,000 "
16,000 "
16,000 "
16,000 "
16,000 "
Train load 3000 lbs per ft.
8' 7.5' 4.5' 4.5' 4.5' 10.5' 5' 5.5' 5' 3'


Class N
Narrow Gauge Consolidation Engine.
8,000 lbs.
8,000 "
15,000 "
15,000 "
15,000 "
15,000 "
8,000 "
8,000 "
8,000 "
8,000 "
Train load 2000 lbs per ft.
6' 6.5' 4' 3' 1' 8' 4' 5.5' 4' 6'

Under these specifications, the weights of bridges given in the following tables have been calculated with every attention to detail, and with a view of obtaining the most economical bridge in each case.

The aim has been to adopt a first-class construction, without any additions of doubtful utility, so that while a bridge might weigh more than that given through some special excellence, it should not weigh much less. This remark should be limited to spans under 300 feet, however, as above that the individuality of engineers becomes more marked, and so much difference of opinion exists as to the proper widths, etc., of bridges, that weights based on one kind of construction would be, in a measure, approximations for another kind.

The formulas herein proposed are intended to give the total or shipping weight of iron or steel in the span.

Some slight sacrifice of exactness has been necessary, in order to make the formulas as simple as possible.

All efforts to combine the total range of spans under one formula failed. It was necessary to have a formula for girders, and a different one for truss spans; or, what seemed more desirable, a formula which would include both girder and truss spans below 225 feet, which are of the most common occurrence, and a different formula for spans over 225 feet.

As far as possible the same style of formula has been maintained.

The type of bridge assumed is as follows: For spans below 75 feet, deck-plate girder bridges, 8 feet wide, connected with angle-iron bracing, and with cross-ties resting on the top chords; above 75 feet, through truss bridges, Pratt or single quadrangular trusses up to 150 feet, and Whipple or double quadrangular trusses over that length.

The widths assumed have been: For standard gauge spans under 255 feet, 14 feet in the clear; for 320 feet span, 18 feet center to center of trusses; for 420 feet span, 21 feet center to center; and for 520 feet span, 25 feet center to center. The floors of the spans consist of cross floor beams at the panel points, with a line of iron stringers under each rail, except for spans over 300 feet, which have three lines of stringers.

Differences in depth affect the weight less than would be supposed. Thus in a 60-foot girder span, for Class T, the difference in total weights between depths of 5 feet and 5½ feet was practically nothing, and for an 80-foot girder span, calculated for Class C, with depths of 6 feet, 6½ feet and 7 feet, the difference was less than 1 per cent. In a 180-

foot truss span, the difference in weights between depths of 26 and 28 feet was less than 2 per cent.

In a 520-foot steel span, for Class T, the difference in weight between a depth of 50 feet and one of 58 feet was about 3 per cent.; a depth of 56 feet was finally taken in this case.

The conditions on which the formulas are based have been stated. Modifications for other conditions may be made as follows:

If wooden stringers are used, deduct 195 pounds per foot for Classes M and C, 210 pounds for Class T, and 140 pounds for Class N.

For safety stringers add 100 pounds per foot for all classes.

For deck truss bridges add 10 per cent., and for double track bridges 90 per cent. to the formula weight.

Through plate girder bridges will not differ materially from deck bridges in weight, where the cross-ties are made to serve as floor beams; where, however, an iron stringer floor is used, it will be a close approximation to add 200 pounds per foot to the weight as given by the formula.

The following are the formulas proposed, in all of which

S = the span center to center of bed plates or end pins, as the case may be.

W = the total or "shipping" weight of iron or steel in pounds.

For iron bridges under 225 feet span:

$$W = \left(75 + \frac{S}{a}\right) S\sqrt{S} \qquad (1)$$

in which

$a = 7$ for Class T.
$a = 9$ " " C.
$a = 12$ " " M.

For Class N take three-fourths of the weight as given for Class T.

Table No. 1 is for this formula. In it are given the actual or estimated weights, and the corresponding weights by formula, with percentages of differences, taking the actual weight as unity.

TABLE No. 1.—For Iron Bridges under 225 feet span.

Span in feet.	Class M, Moguls and 1 820 lbs.		Per Cent. of Difference.	Class C. Consols. and 2 240 lbs.		Per Cent. of Difference.	Class T. Typ. Consols. and 3 000 lbs.		Per Cent. of Difference.	Class N. N. G. Consols. and 2 000 lbs.		Per Cent. of Difference.
	Actual.	$\left(75+\frac{S}{12}\right)S\sqrt{S}$		Actual.	$\left(75+\frac{S}{9}\right)S\sqrt{S}$		Actual.	$\left(75+\frac{S}{7}\right)S\sqrt{S}$		Actual.	$\frac{3}{4}\left(75+\frac{S}{7}\right)S\sqrt{S}$	
20	6 841	6 856	$+ 0\frac{2}{10}$	6 841	6 901	$+ 0\frac{9}{10}$	7 047	6 960	$- 1\frac{2}{10}$	5 794	5 260	$- 9\frac{2}{10}$
30	12 523	12 741	$+ 1\frac{7}{10}$	12 732	12 872	$+ 1$	12 979	13 036	$+ 0\frac{1}{2}$	9 576	9 828	$+ 2\frac{6}{10}$
40	18 635	19 794	$+ 6\frac{2}{10}$	19 283	20 072	$+ 4$	19 807	20 403	$+ 3$	14 832	15 303	$+ 3\frac{2}{10}$
50	27 639	27 997	$+ 1\frac{3}{10}$	28 561	28 456	$- 0\frac{4}{10}$	29 115	29 022	0	22 303	21 765	$- 2\frac{4}{10}$
60	37 360	37 200	$- 0\frac{4}{10}$	38 590	37 944	$- 1\frac{7}{10}$	39 502	38 874	$- 1\frac{7}{10}$			
70	48 739	47 348	$- 2\frac{9}{10}$	49 983	48 462	$- 3\frac{1}{10}$	51 033	49 810	$- 2\frac{5}{10}$			
*80	63 061	58 431	$- 7\frac{3}{10}$	64 561	60 005	$- 7$	66 061	61 793	$- 6\frac{5}{10}$			
104	90 555	88 788	$- 2$	93 050	91 812	$- 1\frac{3}{10}$	96 957	95 312	$- 1\frac{7}{10}$	70 464	71 484	$+ 1\frac{4}{10}$
130	124 871	127 200	$+ 2$	129 688	132 380	$+ 2$	137 285	138 850	$+ 1\frac{1}{10}$	99 638	104 145	$+ 4\frac{6}{10}$
150	154 427	160 781	$+ 4$	163 302	168 315	$+ 3$	173 849	177 190	$+ 2$	133 693	132 891	$- 0\frac{6}{10}$
175	204 486	207 441	$+ 1\frac{1}{2}$	220 006	218 559	$- 0\frac{7}{10}$	232 637	231 525	$- 0\frac{1}{2}$	173 330	173 643	$+ 0\frac{1}{10}$
201.5	267 410	262 548	$- 1\frac{8}{10}$	286 055	278 500	$- 3$	304 459	296 300	$- 2\frac{6}{10}$			
†255	441 065	391 755	$-11\frac{3}{10}$	468 884	420 668	$-10\frac{3}{10}$	508 415	453 776	-12			
†320	778 552	581 639	$-25\frac{3}{10}$	837 387	632 590	$-24\frac{5}{10}$	946 031	690 983	$-26\frac{9}{10}$			

* These 80 feet spans are plate girders. Pin connected trusses for this length would weigh less.

† These spans show how much the formula is in error for spans greatly exceeding the limit.

For Iron Bridges over 200 *feet span.*

$$W = \left(5 + \frac{S}{b}\right) S^2 \qquad (2)$$

in which $b = 100$ for Class C.
$b = 80$ " T.

Table No. 2 gives values for this formula.

TABLE No. 2.—FOR IRON BRIDGES OVER 200 FEET SPAN.

Span c. to c. in feet.	Class C. Consols. and 2 240 lbs.		Per Cent. of Difference.	Class T. Typ. Consols. and 3 000 lbs.		Per Cent. of Difference.
	Calculated	$\left(5+\frac{S}{100}\right) S^2$		Calculated	$\left(5+\frac{S}{80}\right) S^2$	
201.5	286 055	284 317	$-0\frac{6}{10}$	304 459	304 567	0
255	468 884	490 900	$+4\frac{7}{10}$	508 415	532 555	$+4\frac{7}{10}$
320	837 387	839 680	$+0\frac{3}{10}$	946 031	921 600	$-2\frac{6}{10}$
420	1 633 244	1 622 880	$-0\frac{6}{10}$	1 829 484	1 808 100	$-1\frac{6}{10}$
516	2 635 617	2 689 185	+2	2 977 191	3 048 631	$+2\frac{6}{10}$

For Steel Bridges over 300 *feet span.*

$$W = c\,S^2 \qquad (3)$$

in which $c = 6$ for Class C.
$c = 6.7$ " T.

Table No. 3 gives values for this formula. The formulas have been framed on independently calculated spans, in order to secure uniformity of conditions. Prototypes of the iron spans of 320 feet and under, with two exceptions, have been built within the past five years, and the formulas have otherwise been checked on over eighty spans built or designed within that time. The weights for steel spans are based on bridges which have been completely designed, but not built. They are of a very high type of construction. A frequent difference between the formula weights and those of existing bridges is caused by the custom which prevailed at one time, of assuming the webs of plate girders to assist the flanges and reducing the sections accordingly.

TABLE No. 3.—For Steel Bridges over 300 feet span.*

Span c. to c. in feet.	Class C. Consols. and 2 240 lbs.		Per Cent. of Difference.	Class T. Typ. Consols. and 3 000 lbs.		Per Cent. of Difference.
	Calculated	$6\ S^2$		Calculated	$6.7\ S^2$	
305.5	553 636	559 980	$-1\frac{1}{16}$	625 600	625 311	0
320	578 520	614 400	$+6\frac{3}{16}$	654 400	686 080	$+5$
374	884 700	839 256	$-5\frac{1}{16}$	944 000	937 169	$-0\frac{7}{16}$
400	974 243	960 000	$-1\frac{5}{16}$	1 073 545	1 072 000	$-0\frac{1}{16}$
420	1 052 720	1 058 400	$+0\frac{1}{2}$	1 182 835	1 181 880	0
450	1 237 540	1 215 000	$-1\frac{9}{16}$	1 382 500	1 356 750	$-1\frac{9}{16}$
475	1 333 830	1 353 750	$+1\frac{5}{16}$	1 481 510	1 511 687	$+2$
516	1 639 426	1 597 536	$-2\frac{7}{16}$	1 773 689	1 783 915	$+0\frac{8}{16}$

* In these spans the floor-beams, stringers, lateral struts, counters and small lateral rods are made of iron, in accordance with the universal practice in this country.

DISCUSSION.

D. J. WHITTEMORE, Past Pres. Am. Soc. C. E.—In reference to this paper, it is pertinent to inquire whether there is such general uniformity of design at the present time among those members of our profession, expert in bridge construction, as to warrant the use of simple formulas in determining the approximate weight of metal that should be employed in their construction. Probably no other problem in mechanical construction has been subjected to such thorough analysis, and by so many able minds, as this of iron bridge design, and if, as a result of such rigid investigation, it is possible to estimate approximately the weight of properly designed structures by some such simple formulas as those given in this paper, I feel assured that to the engineer who has but little expert knowledge of the subject it will be often of great assistance and consequent value, though to the expert it may not afford that close approximation demanded by him.

Any general discussion of this paper should, and I trust will, cover the whole subject of bridge construction; in how far we agree as to standard train loads and wind strains, permitted strains in and forms of parts, and their mode of manufacture—all factors bearing upon the safe and economic weights of structures.

Applying the formulas to several structures, the weights of which are known to me, I find the same to give close approximate results, after deducting the weight of material used for purposes that are assumed by the writer to be of doubtful utility, but esteemed by me highly proper in resisting abnormal strains, to which every structure in constant use is liable.

It would be of interest, could we know whether the weights of structures given were derived from designs made mainly by one, or from designs made by several engineers of acknowledged experience.

I would ask our Members whether it is the generally accepted practice in designing plate girders to consider that the flange shall have sufficient area to bear its stress without assistance of the web, the latter to provide only for shearing stress as borne by the web alone, as stated by the writer.

Perhaps it may be proper, in the discussion of the paper, to call for an expression from our Members as to whether we are warranted in sub-

jecting steel to the strains specified by the writer, with our present limited knowledge of this metal, particularly when called to resist those of compression.

Considering the unusual facilities offered the writer in gathering data, and the labor and thought expended upon the subject treated, I trust this paper will receive that consideration it fully merits.

JOSEPH M. WILSON, M. Am. Soc. C. E.—The paper in question furnishes a valuable addition to what data we already possess on the subject of the determination of bridge weights. The ability to predetermine readily, and with reasonable accuracy, the weight which a bridge conforming to certain standard specifications should have, is a great desideratum. Not only is this so for the engineer, to enable him to assume his preliminary loading for calculation, but in this country, where the deciding of competitions and awarding of contracts is so often in the hands of those who have but little knowledge of the requirements for a good bridge, or what amount of material is necessary to build a bridge according to certain agreed specifications, the advantage of being able to apply readily some simple formula to the case is obvious. These formulas seem to meet the latter requirement particularly.

A paper was contributed some time since to the Institution of Civil Engineers, by Max Am Ende, on the Weight and Limiting Dimensions of Girder Bridges (Proc. Inst. C. E., Vol. LXIV, p. 243), which appears to consider the same subject, with rather a wider scope. Mr. Am Ende submits a general "theoretical formula for the weight, in which the weight of the bridge, the weight of the platform and the weight of the moving load; the principal dimensions and a number of constants, appear as undetermined quantities." The moving load and the weight of the platform being assumed, also the width of the bridge as a function of the span; and the constants being determined from the practice of the designer who constructs the parts of the bridge, the formula gives the desired result. Mr. Am Ende states that almost the only difficulty he encountered in the way of a satisfactory solution of the problem was in the case of secondary bracing.

The formulas of the present paper being derived from actual or calculated weights of known structures, or structures already designed, do not possess the scientific interest of the formula of Mr. Am Ende, and while no doubt giving reliable results within certain limits, yet they

might not be applicable for very long spans where little or no data exists to base them upon. Neither do they take any account of variations in the proportions of the structures. But they possess the great advantages of simplicity and readiness of application, both of which are very important in the uses to which they will probable be most frequently applied.

Should the standard loading for any structure in question not be the same as assumed in one of the cases in the paper, the requisite modification of the value of a can of course be made. In view of the continually increasing weights for engines and train service, the writer is in favor of high standards. The typical engine given in the paper is one which the writer had worked up several years ago, and adopted in specifications for bridges on the lines of the Pennsylvania Railroad, to anticipate what might come in the future. No engine has yet been built on this type, but one of another type, lately put in use, really surpasses it in its effects in certain special cases.

It appears to the writer that the results, as achieved by these formulas, might have been more readily accomplished graphically, by the use of a curve on cross-section paper, the ordinates laid out on the axis of the curve representing the spans of different structures, and the abscissæ the weights, either in pounds per lineal foot, or as desired. As many points for the curve as possible could be fixed from actual data, and the curve constructed thereon. Different classes of structures, such as solid plate girders, open-built girders, highway bridges, etc., could all be represented on the same sheet. The writer has used such a method with success, but while in his opinion better for the engineer, it might not, perhaps, be as applicable for the general public as formulas.

W. R. Hutton, M. Am. Soc. C. E.—Although no one series of formulas giving the relation of weight to span of railroad bridges, is in general use, yet many bridge-builders and experts have some rule by which to determine the weight of the structure for preliminary estimates with considerable accuracy.

The weight of the trusses will vary closely with the square of the spans, the floor system directly as the span and the width between trusses, and the horizontal bracing with the width and with the square of the span.

The specification quoted by Mr. Pegram contains one erroneous formula, although not uncommon in general practice. I refer to the application to a long steel column of the formula for an iron column "with 50 per cent. added." Experiments have shown that the resistance of long columns of steel varies with their length more rapidly than those of iron, approaching that of iron when the ratio of length to diameter becomes very great. This might have been anticipated with a metal whose resistance to crushing is much greater than iron, and its modulus of elasticity nearly the same. Those engineers act more prudently who limit the use of steel in compression to comparatively short parts, where its greater strength is available; using for the longer and generally lighter parts the better known material; for in these members the saving in weight is less, while the uncertainty as to strength is greater.

There is a specification that in plate girders the flange stress shall be borne by the flange alone, the shearing stress by the web alone. This is not unusual, but it would seem to be an unnecessary precaution in view of the low strains generally admitted in the metal of such forms.

The present dull season gives an excellent opportunity to many of our Members to work out a correct formula for strength of columns, in which the modulus of rupture (or the elastic limit) shall be combined with the modulus of elasticity—or at least a formula similar in form to Gordon's—which shall represent result of experiments in steel.

W. M. HUGHES, M. Am. Soc. C. E.—Having the shipping weight of a large number of spans constructed from specifications substantially the same as those given in this paper, using two class "M" engines, one followed by the other, and this by a freight load of 2 000 pounds per lineal foot, I have applied the formula to a few of them with the result shown in the following table.

The iron stringers for 15-foot panels weighed about 160 pounds per lineal foot, a $\frac{5}{16}$ inch web plate being used. Where wooden stringers are used, I have deducted 195 pounds per lineal foot from result of formula as suggested by the author. As will be seen, the percentage of difference here in each case is minus. These are all "through" bridges, the last two being double track with two trusses. It will be seen that these weights are less than 60 per cent. added to the result of the formula, instead of 90 per cent. as suggested by the author of the paper.

It would seem that the difference here shown between the actual weights and those given by the formula are too great to make it of much practicable use.

TABLE No. 4.

Span.	Kind of Truss.	Stringer.	Depth.	Number and Length of Panel.	Actual Weight.	Weight by Formulas.	Per Cent. of Difference
Feet.			Ft. In.	No. Ft. In.			
50	Pratt.....	Iron	6 3	4 12 6	31 360	27 940	— $10\frac{9}{10}$
61	"	"	8 0	4 15 3	39 294	38 150	— 3
75	"	"	21 0	5 15 0	51 157	52 812	+ $3\frac{2}{10}$
84	"	"	21 0	6 14 0	62 875	63 058	+ $0\frac{3}{10}$
105	"	"	20 0	7 15 0	78 476	90 115	+ $14\frac{8}{10}$
120	"	"	22 6	8 15 0	103 094	112 200	+ $8\frac{8}{10}$
135	"	"	22 6	9 15 0	124 015	135 110	+ $*9\frac{1}{10}$
165	Whipple....	Wood	27 0	11 15 0	165 725	155 975	— $5\frac{9}{10}$
184	"	"	28 0	12 15 4	198 351	191 720	— $3\frac{3}{10}$
200	"	"	28 0	13 15 $4\frac{1}{2}$	240 241	225 240	— $6\frac{3}{10}$
150	Pratt.....	Iron	28 0	7 21 5	243 930	= formula.	+ $51\frac{7}{10}$
142	"	"	27 0	7 20 $3\frac{1}{2}$	235 700	= "	+ $59\frac{7}{10}$

THEODORE COOPER, M. Am. Soc. C. E.—Every experienced bridge engineer has some method of approximating the weight of a proposed bridge. It may be by the use of formulas, by tables, by diagrams, or it may be by apparent guessing, based upon accumulated experience in similar work. Such methods used by any engineer for his own guidance, if given to the public, would be misleading; or, at the best, simple approximations, for they must be varied for each individual case, according to the specifications and requirements and to suit the particular style of bridge adopted.

The author speaks as if there was but one general standard of specification in use in this country. While in the better class of structures this is getting to be the case, in general terms there is still considerable variation in loadings, details and in the unit strains allowed on the various members. In general, a wider variation exists in the floor system than in the main girders. In some specifications the floor will be based upon a fixed load per lineal foot of track; in others upon some single type of engine; in others, upon the maximum due to several types of engines. In my own specifications I use a standard form of heavy freight engine, and a fixed load on two pair of axles to cover present or probable loads on heavy passenger engines. Some specifications call for four stringers under each track; specifications also vary in

he character of the wooden portion of the floor. In each special case there is an opportunity for variation, requiring the judgment of the expert to adapt the formulas.

The author of the paper speaks as positively of the allowed strains upon steel as in the case of iron bridges. Now, as before remarked, in the better classes of iron construction, experienced bridge engineers are approaching towards a general standard of unit strains for the various parts. But there is no justification for including steel in this statement. Our experience and knowledge of the adaptability of steel for the various parts of a bridge are still exceedingly limited. The assumption of 50 to 75 per cent. increase in unit strains upon steel over the allowed unit strains on iron members is entirely unwarranted as yet, and most certainly cannot be acknowledged as "standard for this country."

I must take decided exception to the inference conveyed in the first two paragraphs of the author's paper: That those engineers who are opposing the use of steel are in the same state of ignorance as those who assert there is no law (if there are any such) governing the variation of weights of bridges. Speaking for myself, I claim to have some "appreciation of the rapid progress being made in engineering;" nevertheless I do oppose the present loose manner of advocating the use of steel for bridges. This is called opposing the use of steel, but it is an entirely different thing. I believe steel to be the coming material of bridges and like heavy structures, but it will not become so until the manufacturers and users prove its capabilities for the particular purposes.

The steel men of the day are following blindly into the same paths that the iron men took when iron bridges were first built—making claims far beyond its capabilities, and using it in untried forms. The leading manufacturers of iron bridges soon found it necessary to determine by actual tests what different forms would best serve the purpose, and what were the capabilities of the material in its different forms. We hear no more of 60 000 pounds iron and flat plates for compression members.

Do the steel manufacturers appreciate that a false step can put back for years their possession of this great field of structural material? The danger is not in our great structures, built by intelligent and conscientious engineers, but from the thoughtless or unscrupulous imitators. Light structures are being built, in which the compression members, made of steel, are reduced to absolutely treacherous limits. Their ad-

vocates justify their proportions upon the loose statement that steel members are 50 per cent. stronger than iron. Now, if we may put this additional load upon tension members and compression members that must fail by crushing, there is no evidence that the same rule applies to members failing by deflection. In fact experiments show that at a point within the limits of actual practice, there is no increase of strength in steel over iron for long compression members.

As engineers, it is our duty to know the capacity of the materials we use, and we have the right to challenge these injudicious advocates for using steel in this manner, for their evidence of the increased strength of steel over iron in all its forms: actual tests, not assumptions.

The PRESIDENT—Gentlemen, you have heard the challenge. Bear it in mind.

CHARLES MACDONALD, M. Am. Soc. C. E.—Have you any such tests in regard to iron?

THEODORE COOPER, M. Am. Soc. C. E.—Yes, sir; we have full tests of all forms of iron—columns, eye-bars, etc., but in regard to steel we have nothing as to compression members, and it is simply presumed that steel, because it is steel—no other reason—is 50 per cent. stronger than iron. Now, if that is science, I fail to know what science is. I have always supposed that science was based upon actual facts, and experiments and deductions therefrom. Now, is it a scientific deduction that because steel is stronger in tension than iron, it is therefore stronger in compression?

CHARLES MACDONALD, M. Am. Soc. C. E.—I should be sorry to see an impression prevail that American engineers were hesitating about the use of steel in bridge construction on account of a supposed lack of experimental information as to the strength or uniformity of that material. As a matter of fact, a very considerable number of tests have been made upon steel eye-bars of varying sizes, and the results prove beyond a question that they are being produced of a quality far superior to the best iron eye-bars. Even Bessemer steel, which has been thought inferior to open hearth in uniformity, is now giving quite as good results as the more favored material. In large sections to resist compression, as in top chords and end posts of bridges, we have no more experimental

information as applied to iron than steel, and we have a right to assume that where flexure does not result from compression, the strain per square inch on full sections in steel may be taken at the same ratio to the unit strain for that material, as would be considered good practice in the case of iron. In long columns, as in intermediate posts, it would be premature to advise the use of steel at present, by reason of the complicating influence of flexure. A comprehensive system of experiments in this direction is urgently needed, not only for steel but iron construction, and it is greatly to be deplored that the Government machine at Watertown, perhaps the only one in the country upon which such experiments could be made, is now fully occupied by what would seem to be very much less important investigation.

The floor system, composed as it is of plate girders, will not be made of steel until the price of steel plates and angles is reduced to an equality with iron, which must come at no very distant date.

THEODORE COOPER, M. Am. Soc. C. E.—My remarks have more of a bearing upon compression members than upon those for tensile strains. We have a very complete series of tests of iron columns of full-sized members. There have been made at Watertown a large number of tests of full-sized compression members, besides those made by Mr. Bouscaren, M. Am. Soc. C. E., and other individual tests.

These all show the reliability of the proportions adopted for our iron bridge columns.

They exceed the theoretical range allowed by Gordon's formula. But the only tests we have of similar forms of steel show less strength than iron columns. Some show only two-thirds the strength of similar ones in iron. I have carefully collected such information as possible as to tests of steel compression members, and I fail to find any justification for considering them as proportionately superior to those of iron. I would like to know from Mr. Macdonald if we are justified in using steel in the top chord of a bridge with 14 000 pounds compression per square inch, why iron of the same size and form would not be as good? We have no evidence to the contrary.

Have you any evidence that iron in the same proportions would not be fully as strong. Is there any fairness in comparing iron bridges with 8 000 pounds on the top chords with steel bridges with 14 000? One or the other is wrong, in my opinion.

Mr. MACDONALD.—I think your point is very well taken.

GEORGE H. PEGRAM, M. Am. Soc. C. E.—Information has been asked on the process of development of the formulas. They are purely empirical; indeed I question the possibility of producing a formula based on theoretical considerations alone, which would be of any practical value. Such a formula would be very complicated and must contain constants, the values of which would have to be supplied from practice, and the formulas themselves checked on the actual weights of finished structures. One important element of uncertainty in a theoretical formula is the influence which the form of the truss has on the weight, and which would not be taken into account in the formula. The common forms of trusses—Warren, Pratt and Whipple—will all weigh about the same at 150 feet span for a single-track bridge. On both sides of this length they will differ. At 175 feet the Whipple truss will weigh 7 per cent. less than the Pratt, making a difference of about 4 per cent. in the total weights of the spans. Above this length the difference will increase.

As stated in the paper, all of the weights given were especially calculated, the weights of built structures and previously prepared estimates being used to check on. Having determined the minimum weights of first-class spans, the development of the formulas was a "cut and fit" process, subject to the conditions that no variable should appear but the span, and that this should be limited to the square and square-root, which may readily be obtained by the engineer from tables at hand, or in any case would be of easy determination.

Mr. Hughes has submitted a very interesting table of weights of structures which have been built.

Those of 50 feet to 84 feet truss spans are particularly interesting in comparison with the plate-girder spans of these lengths given in the paper. In the Whipple spans, if Mr. Hughes had used 160 pounds per foot for stringers, which he gives as the weight for 15 feet panels, the agreement with the formula would have been very close, the 165 feet span differing $2\frac{1}{10}$ per cent., the 184 feet being exact, and the 200 feet span differing but $3\frac{3}{10}$ per cent. The weights, however, are not sufficiently consistent to entitle them to use as a standard, and the conclusions drawn by Mr. Hughes would seem to have no basis in his table. For example, assuming the weight given for the 135 feet span as a basis,

a 142 feet single-track span would weigh 135 000 pounds, to which 75 per cent. must be added to give the weight of the 142 feet double-track span.

Assuming further that a 150 feet single-track span weighs 154 000 pounds, or 4 per cent. less than the formula, which would seem consistent with the other weights, only 60 per cent. must be added to give the weight of the double-track, 150 feet span. We have, therefore, in spans differing only 8 feet in length, 75 per cent. to be added in one case, and 60 per cent. in the other.

Without regard to formulas or tables, it would be a natural inference that a span of double the width to carry double the load should have double the weight, but, in fact, the larger sections, permitting higher stresses to be used, reduce the excess to about ninety per cent.

It is a source of regret that other tables have not been submitted, as in this way the variations that might be expected in the weights of different designs and under different conditions could be determined.

It is fair to ask what discrepancy should be allowed between the estimated and actual weights. In my opinion, the engineers' estimates, generally made before the structure is completely designed, should not be questioned when coming within five per cent. for ordinary spans. In checking on railroad bridges previously built, still greater variations may be expected. Railroads sometimes shorten a standard design without changing the sections, and sometimes lengthen it, increasing sizes, but without a proper reproportionment, in order to expedite the work.

It most frequently occurs, however, that a difference between the actual and formula weights, when checking on existing structures, is occasioned by the use of what has been termed "additions of doubtful utility," referring to stiff end bottom chords used in excess of the sections required, guard angles, devices for guiding a derailed train, etc. It was not meant to question the utility of these parts by what would now appear to be an unhappy expression, as I thoroughly agree with Mr. Whittemore in regard to them. But their use is not sufficiently general to be recognized in formulas such as are proposed, and it was thought best to let them be considered as extras.

Three elements of standard specifications which operate to limit the reduction of weight are as follows:

First.—No compression member shall have a length greater than 48 times its least diameter.

Second.—No plate in a compression member shall have a thickness less than one-thirtieth of the width between rivet supports.

Third.—No web plate shall have a less thickness than $\frac{3}{8}$ inch.

The latter cannot be said to be general, but is so frequently specified in first-class work, that it was observed in preparing weights for the tables.

Regarding steel, strictly speaking there are no standard specifications, but there is a considerable degree of uniformity in its use for the structures which have been built in late years, and which are in course of construction, and with the attention now directed to the subject, something like a standard practice will doubtless soon be developed. In the Plattsmouth and Bismarck Bridges, steel was taken at 15 000 pounds per square inch in tension, and the same in compression, for members under sixteen diameters in length. It was not allowed in long compression members. In other large bridges now in course of construction, steel is taken at 14 000 pounds in tension, the same in compression, for members under sixteen diameters in length, and the usual column formula for iron, with 37½ per cent. added for lengths exceeding sixteen diameters. This might properly be termed present practice. The only difference from the specifications given in the paper is in the case of members over sixteen diameters in length, where I have added 50 per cent. to the iron column formula.

What stresses it would be proper to allow in compression can hardly be determined from the meager data which we now have; but if 14 000 pounds can be used in lengths of sixteen diameters, fifty per cent. added to the iron column formula, which would correspond to a numerator of 12 000 in place of 8 000, would not seem excessive; in fact published tests would suggest that the numerator for the column formula be made the same as the unit stress for short members, as in iron, where we take 8 000 in both cases. And in support of this position I shall refer to the two tests published by Mr. George S. Morison, M. Am. Soc. C. E., in his book on the Plattsmouth Bridge, and the experiments of Mr. James Christie, M. Am. Soc. C. E., on Steel Struts, Paper 286, Vol. XIII of the Transactions of the Society.

What Mr. Christie has termed "hard steel" has 0.36 per cent. of carbon, corresponding almost exactly with the steel used in the bridges referred to.

In treating of bridges we should keep within the limits of the mem-

bers used in bridges, and it is of no importance how the material acts outside of those limits. A length of 16 diameters would correspond to about forty times the radius of gyration, and will not differ much from the usual top chords in steel spans. It is doubtful if members will ever be made less than 30 radii of gyration in length. The upper limit would be 48 diameters, corresponding to about 125 radii. This limit would never be exceeded and rarely reached. The question is: What will steel struts do between 40 and 125 radii in length? From the table given by Mr. Christie on page 265, Vol. XIII, as the results of his experiments, it appears that the excess of strength of hard steel struts over iron at 40 radii is 55 per cent.; at 80 radii, 65 per cent.; and at 120 radii, 52 per cent.

From the two tests made by Mr. Morison for Plattsmouth Bridge, which are especially valuable from the care with which they were made, and the minute description given of them in his published work, it appears that the same results cannot be expected in a riveted member that obtain in the single piece.

The steel was specified to contain 0.35 per cent. of carbon, and to have an ultimate strength and elastic limit of 80 000 pounds and 50 000 pounds per square inch respectively. One column, 34 radii in length, failed at 52 890 pounds per square inch, corresponding to the elastic limit of a tension member, and showed but 32 per cent. more strength than an iron column of the same size and construction, tested at the same time.

The other test was on an iron column of the same lateral dimensions, 9 radii in length, which failed at 46 474 pounds per square inch.

This latter column is shorter than would ever be used in practice, and the first is well within the limits of 16 diameters. It would therefore seem unwise to make a distinction between the stresses in long and short columns until it is developed by tests. It seems probable that new formulas will be required for steel, and that the stresses will be reduced.

Prof. A. J. Du Bois, Jun. Am. Soc. C. E.—While it would be incorrect to assert that there is no law governing the variation of the weights of iron and steel bridges, it would, I think, be safe to say that such law can only be imperfectly represented within very narrow limits by any formula in which the only variable quantity is the length of span. As the weight is affected not only by span, but also by depth, width, panel length, wind-bracing, flooring, and live load assumed, any

formula which assumes all these as constant, must be of very limited application. Mr. Pegram has but imperfectly met a portion of this objection by working out constants for three different cases of loading. Even then he finds it necessary to have a second formula for bridges over 200 feet span. Although his calculated results agree well with the actual, it is not apparent that they would agree equally well for different ratios of depth to span, and they certainly would not for varying floor systems and live load.

It has seemed to me that the preferable and more rational way would be to start with the floor system, and estimate its weight separately, as may be easily and accurately done, in any given case, according to the specification for the bridge in question. The wind-bracing can next be estimated. Then the weight of main girders. Lastly the total weight. A series of formulas, which will enable one to do this, would be free from the above objections. Such a method would enable a very close approximation to be made in any special case, according to the actual conditions of that case; and even for the same span, would be accurate enough to show the influence of any change in panel length, depth, live load, width, or floor system. Such a method would apply to all lengths of span, and to steel or iron, or part steel and iron or wood. I appreciate fully the labor and ability involved in Mr. Pegram's paper, and it is only fair to say that his formula seems to give better results than any of the kind heretofore proposed. I have thought, therefore, that the best criticism would be, to give in this connection a method such as that which I have alluded to.

This method involves the use of only two simple formulas. I have applied it to spans, actually executed, varying in length from 17-foot plate girders, to a 400-foot Pratt truss, with uniformly accurate results. Thus, for Bridge No. 44, Northern Pacific Railroad, by Kellogg & Maurice, 153 feet span, I find the weight 171 513 pounds, as against 158 493 actual, or 8 per cent. excess. For a span of same length in the Quincy Bridge, by Clarke, I find 150 612 pounds, as against 148 592 pounds actual, a difference of only $1\frac{3}{10}$ per cent. excess. Now, here are two bridges of the same span, but of very different construction, and our method takes account of such difference. Again, for span 5 of the Quincy Bridge, $197\frac{1}{2}$ feet span, I find 227 689 pounds, as against 224 400, a difference of $1\frac{4}{10}$ per cent. excess.

For span 1 of the Quincy Bridge, 247 feet span, I find 348 764 pounds,

as against 347 126, a difference of only $\frac{4}{10}$ of 1 per cent. For the Bismarck Bridge, by Morison, 400 feet span, I find by the same method 1 030 800 pounds, as against 975 356, a difference of 5 per cent. in excess. Yet this is part steel and part iron, and of very different construction from the Quincy Bridge. Again, for the Plattsmouth Bridge, by Morison, span 200 feet, I find 269 600 pounds, as against 270 387 pounds, a difference of only 787 pounds; while for one of the plate girders of the Plattsmouth Bridge Viaduct, span 30 feet, I find 3 719 pounds, as against 3 489, a difference of only 230 pounds.

If these examples can be taken as any indication, it would seem that the method leaves little to be desired. I am unable to apply it to any of Mr. Pegram's cases, as the necessary data as to depth, width, panel length, floor system, etc., are not known for the several cases. If he will apply it himself, I anticipate that he will report favorably. The method is neither tedious nor abstruse.

I will now give the formulas and apply them to the cases above cited, and then close by giving the development of the formulas themselves, showing that they are perfectly rational in form.

For plate girders—

Let W = the total external load in pounds.
W_1 = weight of a plate girder in pounds.
l = clear span in feet.
d = depth of plate girder in inches.
r = ratio of span to depth, both in same unit.
S = allowable unit stress, in pounds per square inch.

Also for trusses—

Let w_1 = total live load per foot of truss in pounds, for one truss.
w_2 = weight of floor, ties, planks, rails, spikes, etc.; also of floor beams, floor stringers, and wind-bracing per foot of truss, in pounds for one truss.
w_3 = weight of one truss in pounds per foot.
d = depth of truss in feet.
p = panel length in feet.

Then we have for the weight of a plate girder, consisting of riveted plate and angles, with connections,

$$W_1 = \frac{W l r + 2 d S l}{1.2 S - l r}$$

and for the weight of one truss in pounds per foot run

$$w_3 = \frac{(w_1 + w_2)\,(l - p)\,r + 20\,S\,d}{1.75\,S - l\,r}$$

These two simple formulas will enable us to estimate very closely, and with but little work, the weight of any iron, or iron and steel Pratt truss, double or single intersection, for single or double track, and for any panel length, depth or span.

We may estimate wind bracing by the formula, weight per foot run for wind bracing $= \frac{l}{2} + \frac{1600}{l}$

EXAMPLE 1.—BRIDGE NO. 44, NORTHERN PACIFIC RAILROAD, BY KELLOGG & MAURICE.

Span = 153 feet; depth = 26 feet; width = 15 feet; panel length = 17 feet; $r = 6$.

In this case the weight of ties, planks, rails, spikes, etc. = 370 pounds per foot, or 6 290 pounds per panel.

The floor is supported by plate girder stringers and floor beams.

1. *Weight of iron stringers:*

$l = 17$ feet; $d = 28$ inches; $r = 7.3$; $S = 8\,000$ pounds.

The stringers were calculated for a live load = 51 000 pounds.

Weight of floor =	3 145	"
	54 145	" = W

Hence the weight of one stringer is

$$W^1 = \frac{54\,145 \times 17 \times 7.3 + 2 \times 8\,000 \times 28 \times 17}{1.2 \times 8\,000 - 17 \times 7.3} = 1\,512 \text{ pounds.}$$

1 620 = actual.

2. *Weight of iron floor beams:*

$l = 15$ feet; $d = 36$ inches; $r = 5$; $S = 8\,000$ pounds.

The floor beams were calculated for a live load = 81 600 pounds.

Floor	= 6 290	"
2 stringers =	3 024	"
	90 914	"

Hence the weight of one floor beam is

$$W^1 = \frac{90\,914 \times 15 \times 5 + 2 \times 8\,000 \times 36 \times 15}{1.2 \times 8\,000 - 15 \times 5} = 1\,623$$

actual = 1 728

3. *Wind-bracing* $= \frac{153}{2} + \frac{1\,600}{153} = 86$ pounds per foot; actual = 77 pounds.

4. *Weight of main girder.*—The unit stress assumed is 10 000 pounds, and the live load 2 240 pounds per foot. Hence $S = 10\,000$, $w_1 = 1\,120$.

To find w_2, we have 8 floor beams at $1\,623 = 12\,984$

18 stringers at $1\,512 = 27\,216$

$153)\overline{40\,200}$

263 pounds per foot.

Wind-bracing 86

Floor 370

$2)\overline{719}$

360 pounds $= w_2$

We have then $w_1 + w_2 = 1\,480$; and hence the weight of one main truss is

$$w_3 = \frac{1480 \times 136 \times 6 + 20 \times 10\,000 \times 26}{1.75 \times 10\,000 - 153 \times 6} = 386 \text{ pounds per foot.}$$

$380 =$ actual.

5. *Total Weight of Iron.*—We have now for the two main trusses..... = 772

Wind-bracing..... = 86

Floor beams and stringers..... = 263

1 121

And hence total weight $= 1\,121 \times 153 = 171\,513$

$158\,493 =$ actual.

Difference $= 13\,020 = 8$ per cent. in excess.

EXAMPLE 2.—SPAN 7 OF QUINCY BRIDGE. BY CLARKE.

Span $= 154$ feet; depth $= 22$ feet; width $= 14$ feet; panel length $= 11$ feet; $r = 7$; weight of floor $= 300$ pounds per foot; weight of iron floor beams, 1 000 pounds; $S = 10\,000$; $w_1 = \frac{2\,500}{2} = 1\,250$.

Here, since the entire floor system is given, we have 26 floor beams at $1\,000 = 26\,000$, or 169 pounds per foot.

Wind-bracing $= \frac{154}{2} + \frac{1\,600}{154} = 87$. Hence $\frac{169 + 87 + 300}{2} = 278 = w_2$, and $w_1 + w_2 = 1\,528$. Hence

$$w_3 = \frac{1\,528 \times 143 \times 7 + 20 \times 10\,000 \times 22}{1.75 \times 10\,000 - 154 \times 7.} = 361$$

For the total weight we have then $(722 + 87 + 169)\,154$..... $= 150\,612$ pounds.

Actual..... $= 148\,592$ "

Difference..... $= 2\,020 = 1\frac{3}{7}\frac{3}{6}$ per cent.

EXAMPLE 3.—SPAN 5, QUINCY BRIDGE.

Span = 197⅓ feet; depth = 24 feet; width = 14 feet; panel length = 12⅓ feet; $r = 8.22$.

Same floor as preceding. $S = 10\,000$, $w_1 = 1\,250$, as before. 30 floor beams at 1 000 = 30 000, or 152 pounds per foot.

$$\text{Wind-bracing} = \frac{197.3}{2} + \frac{1\,600}{197.3} = 106 \text{ pounds per foot.}$$

$$\text{Hence } \frac{152 + 106 + 300}{2} = 279 = w_2, \text{ and } w_1 + w_2 = 1\,529.$$

$$w_3 = \frac{1\,529 \times 185 \times 8.22 + 20 \times 10\,000 \times 24}{1.75 \times 10\,000 - 197.3 \times 8.22.} = 448 \text{ pounds per foot per truss.}$$

For the total weight we have (896 + 106 + 152) 197⅓ ..... = 227 689 pounds.

224 400 = actual.

Difference ..... = 3 289 = 1 1/5 per cent.

EXAMPLE 4.—SPAN 1, QUINCY BRIDGE.

Span = 247 feet; depth = 26 feet; width = 14 feet; panel length = 13 feet; $r = 9.5$.

Same floor as preceding. $S = 10\,000$, $w_1 = 1\,250$, as before. 36 floor beams at 1 000 = 36 000 pounds = 146 pounds per foot.

$$\text{Wind-bracing} = \frac{247}{2} + \frac{1\,600}{247} = 130 \text{ pounds per foot.}$$

$$w_2 = \frac{146 + 130 + 300}{2} = 288, \quad w_1 + w_2 = 1\,538, \quad w_3 = 568.$$

Total weight = (1 136 + 130 + 146) 247 = 348 764 pounds.

347 126 = actual.

Difference ..... = 1 638 = 1/10 per cent.

EXAMPLE 5.—BISMARCK BRIDGE, BY MORISON.

Span = 400 feet; depth = 50 feet; width = 22 feet; panel length = 25 feet; $r = 8$.

Floor = 450 pounds per foot = 11 250 per panel.

1. *Weight of Iron Stringers.*—$l = 25$ feet; $d = 30$ inches; $r = 10$; $S = 8\,000$; live load = 50 000. Hence $W = 50\,000 + 5\,625 = 55\,625$, and $W' = 2\,770$ pounds. The weight, as computed from the dimensions, is 2 657 pounds.

2. *Weight of Iron Floor Beams.*—$l = 22$ feet; $d = 40$ inches; $r = 6.6$; $S = 8\,000$; live load $= 100\,000$. Hence $W = 100\,000 + 11\,250 + 5\,540 = 116\,790$, and $W^1 = 3\,281$ pounds. The weight, as computed, is 2 791 pounds.

3. *Wind-bracing* $= \frac{400}{2} + \frac{1\,600}{400} = 204$ pounds per foot.

4. *Weight of Main Truss.*—$S = 10\,000$, $w_1 = \frac{2\,000}{2} = 1\,000$.

15 floor beams at 3 281 = 49 215 pounds.
32 stringers at 2 770 = 88 640 "
137 855, or 345 pounds per foot.

Hence $w_2 = \frac{345 + 204 + 450}{2} = 500$ pounds, and $w_1 + w_2 = 1\,500$ pounds.

Hence $w_3 = 1\,014$ pounds per foot per truss.

5. *Weight of Iron.*—We have then (2 028 + 345 + 204) 400 ..... = 1 030 800 pounds.
Actual ..... = 975 356 "
Difference ..... = 55 444 = 5 per cent.

EXAMPLE 6.—PLATTSMOUTH BRIDGE, BY MORISON.

Span = 200 feet; depth = 30 feet; width = 16 feet; panel length = 25 feet; $r = 6\frac{2}{3}$. Floor = 460 pounds per foot = 11 500 pounds per panel.

1. *Weight of I Stringers.*—Same as before = 2 770.

2. *Weight of Iron Floor Beams.*—$l = 16$ feet; $d = 38$ inches; $r = 5$; $S = 8\,000$; live load $= 100\,000$. Hence $W = 100\,000 + 11\,500 + 5\,540 = 117\,040$; and $W^1 = 2\,005$ pounds.

3. *Wind-bracing* $= \frac{200}{2} + \frac{1\,600}{200} = 108$ pounds per foot.

4. *Weight of Main Truss.*—$S = 10\,000$, $w_1 = 1\,000$.

7 floor beams, at 2 005 = 14 035
16 stringers, at 2 770 = 44 320
58 355 or 292 pounds per foot.

Hence $w_2 = \frac{292 + 108 + 460}{2} = 430$ and $w_1 + w_2 = 1\,430$.

Hence $w_3 = 474$ pounds per foot per truss.

5. *Weight of Iron* —

We have then $(948 + 292 + 108)\ 200 = 269\ 600$ pounds.

$270\ 387$ actual.

787 = difference.

The above examples are sufficient to show the scope, ease and accuracy of the method. Let us now demonstrate the formulas used.

Demonstration of Formulas.

1. *For Plate Girders.*—The moment at center of span for a plate girder is $\frac{(W + W^1)l}{8}$. Dividing by the depth, we get the strain in a flange. Dividing again by S, we get the area of a flange; multiply by 2, and we have the area of both flanges.

Hence, since $\frac{12\,l}{d} = r$

$$\text{Area of flanges} = \frac{(W + W^1)\,r}{4\,S}$$

The weight per foot for iron is found by multiplying by $\frac{1}{3}^0$

Hence, the total weight of flanges $= \frac{(W + W^1)\,lr}{1.2\,S}$

The area of the web is $d\,t$, if t = thickness in inches. The weight per foot is then $\frac{10\,d\,t}{3}$, and

$$\text{Total weight of web} = \frac{10\,d\,t\,l}{3}$$

Add the weight of flanges and web and put the sum equal to W^1 and solve for W^1, and we have

$$W^1 = \frac{W\,l\,r + 4\,S\,d\,t\,l}{1.2\,S - l\,r}$$

Now, in practice, t is never less than $\frac{1}{8}$ inch, and seldom exceeds $\frac{5}{8}$ inch. To allow for connections, let us take $t = \frac{1}{2}$ inch, and we have

$$W^1 = \frac{W\,l\,r + 2\,S\,d\,l}{1.2\,S - l\,r} \quad \text{(I.)}$$

Which is the formula we have used for plate girders. It gives, as we have seen, in every case a very close estimate.

2. *For Main Trusses.*—The total weight on a truss, according to our notation, is $(w_1 + w_2)\,(l - p) + w_3\,l$, because one-half a panel at each end does not come on the truss. The moment at the center of span is then

$$\frac{[(w_1 + w_2)\ (l-p) + w_3\ l].\ l}{8}$$

Dividing as before by d and S, and multiplying by 2, we have for the weight of flanges per foot, $= \dfrac{(w_1 + w_2)\ (l-p)\ r + w_3\ l\ r}{A\ S}$

where A is a constant. In the case of a plate girder A is 1.2, but for a truss with long panels A will be different, by reason of material required for stiffening long struts, and the fact that the area of flanges is not constant.

By comparison with executed examples, we find A to be for a Pratt truss about 1.75.

We have also for the web—weight of web per foot $= C\ d$, where C is a constant. Adding the weight of flanges and web, and equating to w_3 we have

$$w_3 = \frac{(w_1 + w_2)\ (l-p)\ r + A\ C\ S\ d}{A\ R - l\ r} \quad \ldots\ldots\ldots\

determine the weight of a span by a series of formulas involving five distinct and rather laborious operations, which require the exercise of about as much skill as the ordinary detailed calculations. The formulas are admittedly approximations, and contain several constants derived from practice which it is not clear would be constant for all constructions, and the bases of which are not given. It is questionable whether it would not be better to spend more time and determine the exact weight in the usual manner. This will be governed somewhat by the degree of approximation obtained by the use of the formulas.

The examples given are not sufficient to show this. The first, a 153-foot span, for the Northern Pacific Railroad,* differs eight per cent.; the next three are all spans from the Quincy Bridge, in which the weight of the floor is apparently taken from the bridge, the trusses and wind-bracing only being calculated by the formulas. These weights check well. The fourth and fifth examples are from the Bismarck and Plattsmouth Bridges, in which steel chords were used at a stress of 15 000 pounds per square inch, while Prof. DuBois seems to have taken the stress at 10 000 pounds. Their agreement with the actual weights would, therefore, rather disprove the formulas. The only examples which really support them are the three spans of the Quincy Bridge, which, as I have shown, are not completely worked out. The partial check on one bridge built so long ago is not conclusive.

The advantages which Prof. DuBois claims that these formulas have over those which I have proposed, are provision for the variations of the live load, the floor, the wind-bracing, the width, the panel length, and the depth. The present use of two engines followed by a train renders the live load a difficult element to provide for. Equivalent uniform loads for the same train may vary from 6 000 pounds to 3 000 pounds per lineal foot according to the length of span. To use a formula depending on a uniform load, it would be necessary to first determine what this load should be, which I cannot see that Prof. DuBois has attempted. In giving four classes of loading, I have endeavored to meet this requirement, for it is improbable that there will be any case of loading that cannot be brought under one of the classes, and in the case of a very different class of loading, a value for the constant in the formula may be readily determined. There is so much uniformity in the floor, wind-bracing and widths of bridges, that special calculations would seem an unnecessary refinement. The number of

panels within the ordinary limits will not materially affect the total weight. The depth is the only element of those given which would make much difference, and I believe that practice is sufficiently uniform to make it an unnecessary factor of the formula.

Prof. A. J. DuBois, Jun. Am. Soc. C. E. — I do not think that the five distinct operations involved in the method I propose will be found laborious compared to the ordinary detailed calculations. Three of these operations involve the determination of the stringers, floor beams, and wind-bracing. These can be easily and quite accurately determined in accord with any given specification, and as they make up more than half the dead load, it is advantageous to find them separately and not have them included in one formula whose only variable is the length. The weight of main girder is then easily found, and the fifth operation is simply summation of preceding results. This procedure is rational, and the formulas used are strictly rational in form. All of the varying given data have their due weight, and the result holds for the actual case in hand, not for one of four specifications. But two constants have to be determined, by using two given structures, and the method then holds good for single and double-track railroad bridges and for highway bridges as well, for any moving load which may be desired. It seems therefore, preferable to Mr. Pegram's in scope. Mr. Pegram's formulas are purely empirical, and limited by the specifications adopted and the class of structure, as well as by depth, width, panel length, etc.

In the sense that my two constants must be determined for the class of structure under consideration, my formula is approximate, but only in this sense. From two executed examples these constants are easily found. It is, I think, evident that these constants are the same for the same class of structure.

Mr. Pegram undervalues, I think, the examples given. The first two are bridges of the same span, but built under very different specifications. My method gives a close result for each, according to those specifications. Mr. Pegram's would give the same result for both, ignoring their difference. The weight of floor is taken as specified in each case, so also the live load in all the cases. The Bismarck and Plattsmouth Bridges show that it it not necessary to deduce new constants for part iron and steel. It would be out of place for me to determine equivalent uniform loads, because in each case I take an executed structure, assume the load

in accord with which the structure was designed, and compare the result with the actual weight. Mr. Pegram may assume any load he pleases, and my method will give him the corresponding weight in accordance with any practice as to floor system and wind-bracing. There is not, I think, such uniformity in depth as to make a formula which ignores it reliable in all cases, still less such uniformity in floor system and wind-bracing.

Mr. Pegram's method would require a series of formulas and constants to fit it to all the cases which mine easily covers. But I do not wish to be understood as undervaluing his formulas. Properly used within the limitations proper, they appear to be the best empirical formulas yet given. The objections I have stated are those common to all formulas of this character, and the advantages claimed for my own are those possessed by all rational formulas.

For a single-track railway Pratt truss, executed in accord with Mr. Pegram's specifications, his formulas will undoubtedly give readily a sufficiently accurate weight. Outside of those specifications, and especially for other classes of railway bridges, and for highway bridges, they have no application. I should be satisfied if, in this supplementary way, my method may be found serviceable, and if, as I have suggested, Mr. Pegram should think it worth while to test my method by the numerous cases he has already worked out, the necessary data for which are not available to me.

GEO. H. PEGRAM, M. Am. Soc. C. E.—I have checked Prof. Du Bois' formulas by the data on which my formulas are based with results as shown in the following tables:

TABLE No. 5.—For Plate Girders.

Span.	Depth.		Total Load, Exclusive of Girder.	Formula Weight of One Girder.	Actual Weight of One Girder.
Feet.	Ft.	In.			
*20	2	0	68 700	2 110	2 645
*30	3	0	96 000	4 594	4 871
40	3	6	110 000	7 772	7 818
40	4	0	110 000	7 554	7 824
50	4	6	130 000	11 786	11 835
50	5	0	130 000	11 550	11 822
60	5	0	144 000	16 714	16 432
60	5	6	144 000	16 307	16 614
70	6	0	156 870	21 962	21 117
80	6	0	172 000	29 446	28 535
80	7	0	172 000	28 097	28 114

* Without stiffeners, as these spans are sometimes built for stringers, the formula weight would be very close.

All weights and loads are in pounds. The stress is the average of the stresses for top and bottom flanges = 9 000 pounds. The live load is the equivalent uniform load for class T, with percentage for impact added as required by the specifications.

TABLE No. 6.—For Iron Trusses.

Span.	Depth.	Panel.	Stress.	Class of Live Load.	Load Per Foot, Exclusive of Truss.	Formula Weight of One Truss.	*Actual Weight of One Truss.	Per Cent. of Difference
Ft. In.	Ft. In.	Ft. In.						
104 0	24 0	17 4	8 800	*M*	1 820	34 112	23 658	+ 41 $\frac{6}{10}$
				C	2 000	34 632	24 458	
				T	2 090	34 798	25 074	
150 0	25 0	16 8	8 800	*M*	1 675	59 400	46 032	+ 22 $\frac{4}{10}$
				C	1 844	60 750	49 628	
				T	1 950	61 650	52 627	
201 6	28 0	16 9½	8 800	*M*	1 565	101 233	85 131	+ 12 $\frac{4}{10}$
				C	1 776	105 283	93 626	
				T	1 946	108 608	100 283	
320 0	34 0	20 0	9 000	*M*	1 605	267 680	273 671	— 6 $\frac{2}{10}$
				C	1 798	281 088	299 718	
				T	2 088	301 632	302 206	
255 6	29 0	18 3	8 900	*T*	1 943	168 364	185 558	— 9 $\frac{3}{10}$
	38 0	18 3	8 900	*T*	1 943	181 405	174 945	+ 3 $\frac{6}{10}$

* These weights do not include bed-plates, end-shoes and rollers, which for one truss will be: for 104 feet span, 1 800 pounds; 150 feet span, 2 000 pounds; 201½ feet span, 3 400 pounds; 255½ feet span, 3 900 pounds; and 320 feet span, 4 500 pounds. It is not clear whether Prof. Du Bois wishes these included, otherwise he makes no provision for them.

The formulas admit of much readier application than would be inferred from their complex appearance. The results for girders are all that could be asked. For trusses the formula gives very good approximate results for spans over 200 feet; but neither the girder nor truss formula gives proper variations for different depths and loads.

For example, an 80 foot plate-girder 7 feet deep, actually weighs 421 pounds less than one 6 feet deep, whereas the formula difference is 1 349 pounds; and a 255½-foot span, 38 feet deep, should weigh, by formula, 13 041 pounds more than one 29 feet deep, whereas it actually weighs 10 654 pounds less, a total difference of 23 659 pounds, or about 13 per cent.

As regards loads, the difference in weight between a 201½-foot truss for class T and one for class M, is by formula 7 500 pounds, while actually it is double that amount.

The 255½-foot span has been completely worked out for panels of 18.25 feet, 21.3 feet, and 25.5 feet respectively, and the greatest difference in total weights found to be about 1½ per cent.

The formula for bracing gives very good approximate results, except in girder spans, where any excess may be due in a measure to the angle-iron bracing. On the whole, I believe the formulas will be found very useful and sufficiently exact for the purposes for which they will generally be used, except in the trusses under 200 feet span, which, I should think, might be brought into line by some modification.

They do not, however, possess the advantages over mine which Professor DuBois claims in variations for depth, live load and panel length, and in conclusion, I would repeat what I have said on a former occasion, that the panel length will not materially affect the total weight, and there is not enough difference in practice in widths and depths to require that they should be factors in a formula for total weight.

J. S. Elliott, C. E.—Through the courtesy of Mr. A. M. Wellington, M. Am. Soc. C. E., who knew that I was collecting data on this subject, the advance proof of Mr. Pegram's article was kindly sent to me for purposes of discussion by Mr. John Bogart, Secretary of the Society. Mr. Pegram, by taking standard specifications, and by using for his three given span divisions precisely those types that both theory and practice unite in pronouncing most suitable, practically disarms criticism, and limits a discussion of his article rather to a checking of results and an

application to existing data. My object, therefore, is to see how applicable these formulas, derived from calculations of what weights should be according to the high standard just explained, may be to many kinds of railroad-bridge types built and in place.

Mr. Pegram's stipulations as to wooden stringers and deck trusses are taken into account in these comparisons.

The formula for class M, $W = \left(75 + \frac{s}{12}\right) s \surd s$ corresponds more closely than any other with the run of specifications of my different types. For the sake of condensation, only the variations in the form of a percentage are tabulated.

TABLE No. 7.—BRIDGES UNDER 200 FEET IN SPAN.

TYPE.	SPAN.	Variation from Actual Amount as a Percentage.	TYPE.	SPAN.	Variation from Actual Amount as a Percentage.
	Ft. In.			Ft. In.	
Plate Girder	23 0	+ 10	Pratt Ribbon	119 3	+ 2.7
" "	39 11	— 3	Through Pratt	126 0	— 7
Deck Pratt	44 4	— 8	Deck "	137 0	+ 1.3
Queen Post	50 0	+ 4.3	" Fink	150 0	+ 2.9
Deck Fink	50 0	— 7	" Warren	150 0	+ 2.6
" Pratt	61 0	+ 17.6	Through Pratt	153 0	+ 4.5
" "	62 6	+ 18.5	" Whipple	153 4	— 3
" Fink	90 2	+ 12.8	" "	153 4	+ 2
" Pratt	78 6	+ 16.4	" Pratt	154 0	+ 13
Through Pratt	80 0	— 7.6	" Whipple	162 3	+ 1.8
Deck "	98 8	+ 9.7	" "	170 0	+ 1.5
" "	99 9	+ 4.4	Deck Fink	180 0	— 4.3
" "	100 0	+ 9.2	Whipple Ribbon	181 6	— 15.5
" "	108 0	— 0.8	Through Whipple	187 6	— 2.4
" "	110 0	+ 5.3	Whipple Ribbon	189 0	— 11.8
Through "	111 0	— 1.5	Deck Whipple	195 0	— 2.4
Deck Bowstring	113 0	— 7	Through "	200 0	— 2.8
" Pratt	118 0	+ 9.5	Deck Fink	200 0	+ 1

Plus in the percentage column means that the formula gives in excess by the noted percentage; the reverse is the case when the minus sign is used.

This tabulated comparison gives 6½ per cent. as the average variation, but if we throw out, in conformity with Mr. Pegram's method, all truss types under 80 feet in span, and also the 154 foot through Pratt truss, as belonging to a railroad bridge built some twenty years ago, we reduce the variation to 5 per cent—a most gratifying result. Mr. Pegram's idea of adding 10 per cent. in case of deck types, is new to me. If anything, it would have been with me a case of subtraction. Small deck trusses

especially are narrower and shallower than through trusses of the same span. This means less material in floor beams, lateral bracing-posts and braces; but, of course, more in the chords. In the balancing, it would seem, then, that the result is 10 per cent. in favor of the deck variety. The few examples I have given in the tables seem to bear out Mr. Pegram in his opinion. The plus and minus variations for this type are pretty evenly divided. The rejection of the 10 per cent. addition would have made the percentage of error much larger and invariably a negative quantity.

Mr. Pegram has given a complete refutation to Mr. Stoney's dictum concerning the possibility of getting formulas applicable to bridge weights, and I regret not having at hand sufficient data to follow him in a suitable comparison with his results for double-track trusses and spans greater than 200 feet. Reliable data seem to be the hardest things to get quickly. It may not be entirely foreign to the subject to suggest that viaduct weights be given a like consideration. Working up some 34 viaduct quantities taken from the Cincinnati Southern Railroad Reports, I found that $W = (S + \Sigma)\ 250$, in which S = total length of viaduct in feet center to center, Σ = sum in feet of profile bent heights from top of masonry pedestals to top of columns, and W = weight of metal in pounds, gave variations of error, where 12 per cent. was a maximum, and 6 per cent. a mean. My formula gave 250 000 pounds less than the sum of actual amounts in the 34 structures (10 150 000 pounds) a variation less than 2½ per cent. These Cincinnati Southern Railroad Viaducts are of the ordinary American type, bents, mostly 30 feet apart, sometimes 60 and 45 feet. The most of them have about 80 feet as a maximum vertical height; the highest has 128 feet, and a significant variation error of —11 per cent., showing the necessity of a change in formula to suit great heights. The Kinzua Viaduct, for example, with its 302 feet maximum height, fell 33 per cent. short.

With more data, I think that an equation could be found to suit all sizes.

GEO. H. PEGRAM, M. Am. Soc. C. E.—I would ask Mr. Elliott whether he does not mean that Σ shall be the sum of the vertical heights of profile taken at some regular distance apart, regardless of what the spans of the trestle may be?

The formula assumes that the weight of bent per vertical foot is the

same as the weight of span per lineal foot. The span will actually weigh about 50 per cent. more per foot than the bent, but in trestles of the usual proportions it so happens that $\Sigma = 2\ S$ nearly, whence the shortage of one is made up by the overplus of the other, and the formula gives very good results.

In checking, I generally obtained very good results, except in low trestles or those with a short and deep profile. The formula is very simple, and in its present form would answer most purposes; but I would suggest that probably more latitude would be obtained with a modified form, viz.:

$$W = (3\ S + 2\ \Sigma)\ 110,$$

in which S, as before, is the total length of trestle, and Σ the sum of vertical heights at 30 feet distances, regardless of the length of span.

Mr. Elliott's extension of the scope of the paper to trestles suggests the idea of including draw-bridges as well. I think it will be sufficient to say that the total weight of a draw-bridge, including turntable, wheels and machinery to turn by hand, will be the same as that of a fixed span of the same total length, and proportioned for the same live load. I have tried this on a number of spans, from 150 feet to 400 feet, both single and double track, and find it remarkably exact.

W. H. Burr, M. Am. Soc. C. E.—Although the formula of Mr. Pegram is not at all a complicated one, it is probably possible to obtain another of a simple character, equally accurate, and possibly more rational. The floor system of stringers and floor beams is nearly a constant quantity for all ordinary spans, and any rational formula for the weight of such spans should contain a constant coefficient multiplied by the span. Below 250 feet of length, truss spans are also known to vary nearly as the square of the span. Hence, some such expression as Eq. (1) might be expected to give very closely the total weight of a bridge.

$$a\ S^2 + b\ S = W \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ (1)$$

a and b are constants, and S and W are the span in feet and total weight in pounds respectively.

A single illustration only of the application of this formula will be given.

The actual and computed weights of Mr. Pegram's "Class T" spans are given in the following table.

In this case $a = 6.25$ and $b = 260$.

TABLE No. 8.

Span.	Actual Weight in Pounds.	$6\frac{1}{4}\ S^2 + 260\ S.$	Per Cent.
Feet.			
20	7 047	7 700	+ 9.4
30	12 979	13 425	+ 3.6
40	19 847	20 400	+ 3.0
50	29 115	28 625	— 1.6
60	39 502	38 100	— 3.7
70	51 033	48 825	— 4.2
80	66 061	60 800	— 8.0
104	96 957	94 640	— 2.4
130	137 285	139 425	+ 1.6
150	173 849	179 625	+ 3.2
175	232 637	236 907	+ 1.8
201.5	304 459	306 159	+ 0.6
255	508 415	472 707	— 7.0

The different percentages for the plate girders are a little larger than those of Mr. Pegram, but for the remainder of the series they are somewhat less, with the advantage of a little more extended application. If the 60, 70 and 80 feet spans were lattice girders, as they should be for an economical construction, the variations would not exceed 3.2 per cent. for spans from 40 to 225 feet. Above the latter limit a break occurs in any simple formula that can be devised.

These few observations are made rather to draw attention to this simple formula than to cover the whole question.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

321.

(Vol. XV.—February, 1886.)

AN ACCOUNT OF SOME OBSERVATIONS OF STREET TRAFFIC.

By CAPTAIN FRANCIS V. GREENE, M. AM. SOC. C. E.

READ DECEMBER 16, 1885.

The object of this paper is to explain certain observations taken recently under the writer's direction, with a view to determining the actual and comparative traffic on the streets of certain cities; and also to call attention to the desirability of having the traffic systematically measured at frequent intervals, and on a uniform system, by the engineers having charge of the pavements in the various cities. In the twenty cities in this country which have a population of over 100 000 each (6 200 000 in the aggregate) the pavements represent an investment of about $250 000 000. With the single exception of railroads, it is doubtful if any other distinct class of engineering works represents so large an outlay. Whether or not these pavements properly fulfill the object for which they were designed, is a question of very great importance, affecting not only

the health and comfort of the residents of the cities, but also the whole subject of intra-urban transportation. This is a matter of dollars and cents, and a great many of them; for with bad pavements, where the resistance to traction and the wear and tear of vehicles and animals are great, the cost of transportation is often double what it is over good pavements, where the resistance to traction and the wear and tear are comparatively small.

Yet, although the pavement question affects directly such large property interests, I am not aware that any systematic observations have previously been taken in this country to determine the nature and volume of the traffic to meet which the pavements are constructed. In Europe such observations have been taken, though not on a uniform system. In the spring of 1873, Colonel Haywood made his well known investigation on "Accidents to Horses" in London, and for this purpose he was obliged to record the traffic. His observations were confined to five streets, Cheapside and The Poultry, Cannon, King William and Gracechurch streets, in the immediate vicinity of the Bank of England and London Bridge. The total length of streets observed was about one mile, and the number of stations fifteen. The observations were not simultaneous, but were scattered over a period of two months. Some of the stations were observed only one day, others two and three days, and some of them six days, which was the maximum. The observations were all made by the police, and were continuous from 8 A.M. to 8 P.M. There were two men at each station, and they observed for three hours consecutively, and were then relieved by others. Each man had two tours of three hours each, or six hours in all, each day. The exact manner in which the record was kept is not described in Colonel Haywood's report* further than the statement that it was kept in books specially prepared for the purpose.

Observations have also been taken on other streets in London and in Liverpool, but the method pursued in taking them I am unable to state. In Mr. G. F. Deacon's paper on "Street Carriageway Pavements," read before the Institution of Civil Engineers in 1879, he refers to certain streets of Liverpool having a traffic of so many tons per annum per yard of width, but he does not state how these data were obtained. And in

* Report to the Honorable the Commissioners of Sewers of the City of London on the Accidents to Horses on Carriageway Pavements. By William Haywood, Engineer and Surveyor to the Commission. London: Charles Skipper & East, Printers, St. Dunstan's Hill. 1873.

Mr. O. H. Howarth's paper on "Wood as a Paving Material under Heavy Traffic," read at the same time and place, he gives a tabular statement of the traffic on twelve principal streets in various parts of London. His unit for comparison is the tonnage per day per foot of width, which is evidently convertible into Mr. Deacon's unit by multiplying by 940. Mr. Howarth states that his observations were made in the month of November, 1878, and on every alternate half hour during the sixteen hours between 7 A.M. and 11 P.M. Further than this he gives no details.

In Mr. G. H. Stayton's paper, read in 1884, on "Wood Pavement in the Metropolis," is given a list of twenty-four streets (including those given by Mr. Howarth) with their traffic. He adopts as his unit the tonnage per yard of width in a day of sixteen hours, but does not state how the record was made.

In Paris, a series of careful observations were taken at 302 different localities between May 1, 1881, and April 30, 1882. The observations were continuous, night and day, for seven days at each place. Nearly all the places were intersections of streets, and the travel was counted both ways. The report does not state the manner in which the tally was kept. It is signed by MM. Allard and Barabant, the Chief Engineers of the two divisions of the city. The figures are very large, running as high as 33 000 vehicles and 42 000 horses at one place. No tonnage is given, the standard of comparison being the number of horses per meter of width.

The observations above noted—those of Haywood, Deacon, Howarth, Stayton and Barabant—comprise all that I have been able to find on this subject in print.

The observations which I shall now describe, were made under my direction during the months of October and November, 1885, by the employees of the Barber Asphalt Paving Company, which has an office and works in ten large cities; viz.: New York, Philadelphia, Chicago, Boston, St. Louis, New Orleans, Washington, Buffalo, Louisville and Omaha. In the arrangements for taking the observations, two objects were kept in view: First, to leave as little as possible to the judgment of the observer; and second, to make the record permanent, so that it could be preserved for examination in all its details at any time. For this purpose I drew up a blank form, a copy of which is here inserted.

TRAFFIC RECORD.

CITY.	STREET.	DATE.	OBSERVERS.	LOCATION OF OBSERVER.
.....	.....	188	.....	.....

HOURS.	VEHICLES.				ACCIDENTS.				REMARKS.
	Less than 1 ton.	Between 1 ton and 3 tons.	Over 3 tons.	TOTAL.	Falls on knees.	Falls on haunches.	Complete falls.	TOTAL.	
7 to 8.....									
8 to 9.....									
9 to 10.....									
10 to 11.....									
11 to 12.....									
12 to 1.....									
1 to 2.....									
2 to 3.....									
3 to 4.....									
4 to 5.....									
5 to 6.....									
6 to 7.....									
Total.....									

Width between curbs.....feet. Grade of street..... .....feet in a hundred $\frac{1}{100}$
State of weather .....
Average temperature on pavement.....° Fahr. Length of street on which accidents are recorded.....feet.
Average speed of light vehicles.....miles per hour. Average speed of heavy vehicles.....miles per hour.

Personally appeared before me....., a Notary Public, in and for the.....of ....
in the State of....., the above named.....and....., who being by me duly sworn, do hereby certify, under oath, that the above record is correct to the best of their knowledge and belief.

.....188 .....
....., Notary Public. .....,

TRAFFIC RECORD.

.....City.
.....Street.
.....188 .
.....Observer.

INSTRUCTIONS FOR KEEPING TRAFFIC RECORDS.

1st. Ordinarily two observers will be necessary to keep the record, in order that it may be uninterrupted during a period of twelve hours. They will divide the time between them according to their convenience, but should change with each other at the end of any given hour, and not during the hour. The observer should be stationed, if possible, in the second story window of a house from which he can have a clear view of the street for at least two hundred yards in each direction.

2d. The observer will be provided with an ordinary recording conductor's bell punch and a sufficient number of record slips in three colors, viz., white, red and blue, which will be used as follows:

WHITE, for vehicles less than 1 ton in weight.

RED, for vehicles between 1 and 3 tons in weight.

BLUE, for vehicles over 3 tons in weight.

The proper slip will be punched as each vehicle with its estimated weight passes the observer's station.

3d. In estimating weights, the following rules will be followed as a guide:

1-horse carriages, empty or loaded. 1-horse wagons, empty or light-loaded. 1-horse carts, empty.	LESS THAN 1 TON. WHITE SLIP.
1-horse wagons, heavy-loaded. 1-horse carts, loaded. 2-horse wagons, empty or light-loaded.	BETWEEN 1 AND 3 TONS. RED SLIP.
Wagons and trucks drawn by 2 or more horses, and heavy-loaded.	OVER 3 TONS. BLUE SLIP.

Special note will be made, in the column of remarks, of any unusually heavy loads, such as 6-horse trucks loaded with stone or iron, and an estimate given of their weight.

4th. Accidents will be recorded in the middle column of the proper slip, as follows:

WHITE, for partial falls on knees.

RED, for partial falls on haunches.

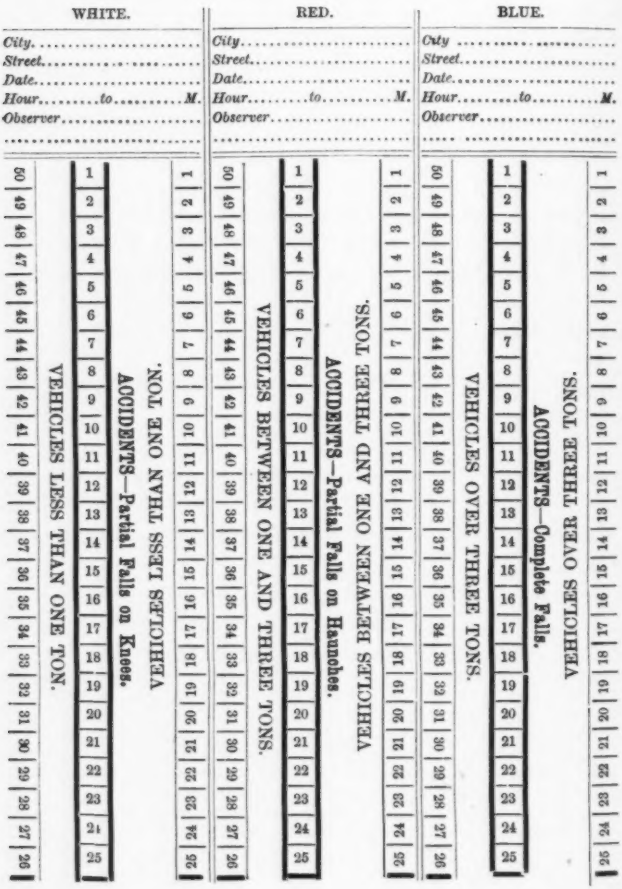
BLUE, for complete falls, in which the horse loses his legs entirely and falls on his side.

All accidents will be recorded which occur within a convenient distance of the observer's station—say to the end of the adjacent block on each side; and the total distance within which accidents are observed will be noted in the report.

5th. The state of the weather, condition of the pavement as to cleanliness, width of roadway between curbs, and location of the observer, should be noted on the report, together with a brief statement of any remarkable or unusual facts connected with the traffic.

6th. Each observer will make up his report on this form, giving the result of his observations, and mail it to Capt. F. V. Greene, West Point, New York, within twelve hours after the conclusion of each day's observations. The punched slips will be carefully preserved in the office of the agent until called for.

For making the actual count of vehicles, I thought the simplest and surest method would be to adopt the trip slip and punch system used by conductors of street cars for counting passengers. Three kinds of slips were printed, all of the same form, but each in a different color, and with different wording for the three classes of traffic and of accident. These slips are shown here.



The traffic was recorded in each city on these slips and blank forms, all of which have been preserved.

The printed instructions on the back of the record blank explain so fully the manner in which the record is kept, that it is hardly necessary to add anything to it.

The traffic is divided into three classes, light weight (less than one ton), medium weight (between one and three tons), and heavy weight (more than three tons); and in order to reduce the personal equation of the different observers to a minimum, the directions specify what classes of vehicles are to be counted in each class of weight. Nothing is then left to the observer's judgment and estimation except the question of "heavy" and "light" loads in one-horse and two-horse wagons. The result of different estimation in this respect between two observers would simply change a portion of the vehicles by one class, and the error in the final result could hardly exceed five per cent.

It will be noticed that the record includes the width and grade of the street, the length over which accidents are observed, the condition of the weather and the speed of vehicles; and that each day's record is signed and sworn to.

The agent in each city was instructed to select the three streets in that city paved with stone, asphalt and wood (if any existed), which, by common report, had the heaviest traffic in the class of pavement used on that street. For the purpose of making certain special comparisons, other streets were selected and their traffic recorded after the first reports were received. The record was in every case made on six consecutive days (Sundays omitted) at the same place, and it was continuous from seven A.M. to seven P.M., except when darkness prevented. No addition was made for this omission, no record was kept during the night, and no addition was made as an estimate of night traffic. In practice one observer was found sufficient on streets of very light traffic, and three (two of whom were always on duty) on the heaviest traffic, such as Broadway; on the majority of streets two observers were necessary, who relieved each other at the end of an hour or two hours.

These reports were all sent to me as soon as completed, and the results have been compiled by myself. In doing so I have discarded the weight of the horses altogether, not because they do not constitute a factor in the wear of the pavement, but because they were discarded in the English reports, and I desired as far as possible to make comparisons

with them. The addition that would have to be made if the horses were included would vary with the traffic. On streets where light vehicles predominate (as on Fifth avenue), it would be about eighty-five per cent.; on streets with heavy vehicles (such as Wabash avenue, in Chicago), it would only be about forty per cent.; and for other streets it would be between these two limits.

In digesting the reports the average daily traffic was obtained by dividing the total record for six days by six. To obtain the tonnage, I estimated the light-weight vehicles to average one-half ton each (including their loads), the medium weight two tons, and the heavy weight four tons. Multiplying the daily average of vehicles in each class by these figures and adding together the products, the total tonnage was obtained, and dividing this by the width between curbs, we get the daily average tonnage per foot of width. This is the standard or unit of comparison. It seems to me more convenient than Mr. Deacon's unit of the annual tonnage per yard of width, because the figures are much smaller and more easily compared and remembered, and it is also better than the French unit of the number of horses, because while the horses have their effect in the wear of the pavement, yet it is very much less than the wear due to the vehicles.

In this manner Table No. 1 has been obtained, which is the summary of the results at each locality.

In order to compare these results with those obtained in England, Table No. 2 has been compiled from the papers of Messrs. Haywood, Deacon, Howarth and Stayton. The Paris reports do not give any tonnage, and hence cannot be used for purposes of comparison.

TABLE No. 1.

GREENE ON STREET TRAFFIC.—TRAFFIC RECORD ON CERTAIN STREETS

CITY.	VEHICLES.			TONNAGE.			
	Less than 1 Ton.	Between 1 Ton and 3 Tons.	Over 3 Tons.	Total.	Total.	Per Vehicle.	Per foot of width	Pav
New York.....	4 379 = 54 per cent.	2 502 = 34 per cent.	930 = 12 per cent.	7 811	10 905	1.39	273	Gran
"	4 957 = 91 "	373 = 7 "	130 = 2 "	5 460	3 744	.68	94	Aspl
"	1 705 = 72 "	551 = 24 "	101 = 4 "	2 357	2 357	1.00	87	Gran
Philadelphia....	3 454 = 58 "	1 499 = 24 "	1 128 = 18 "	6 081	9 237	1.52	142	Aspl
"	3 160 = 63 "	1 409 = 29 "	446 = 8 "	5 075	6 302	1.24	97	
"	1 280 = 70 "	424 = 24 "	110 = 6 "	1 814	1 928	1.06	74	Gran
Chicago.....	907 = 25 "	1 906 = 52 "	824 = 23 "	3 637	7 561	2.08	151	
"	2 244 = 51 "	1 652 = 38 "	493 = 11 "	4 389	6 398	1.46	142	
"	2 368 = 79 "	508 = 17 "	139 = 4 "	3 015	2 756	.90	77	Woo
"	1 697 = 72 "	412 = 18 "	233 = 10 "	2 342	2 604	1.11	69	Aspl
Boston.....	3 774 = 70 "	1 469 = 28 "	119 = 2 "	5 362	5 301	.99	196	Woo
"	3 393 = 69 "	1 400 = 28 "	133 = 3 "	4 926	5 028	1.02	157	Gran
"	2 579 = 73 "	860 = 25 "	64 = 2 "	3 503	3 265	.93	126	Aspl
"	2 849 = 80 "	727 = 20 "	15 = 0 "	3 591	2 938	.80	75	Gran
"	1 244 = 84 "	225 = 15 "	15 = 1 "	1 484	1 130	.79	43	Aspl
"	993 = 89 "	118 = 11 "	3 = 0 "	1 114	744	.67	32	
St. Louis.....	2 258 = 69 "	697 = 22 "	292 = 9 "	3 247	3 691	1.13	103	
"	1 900 = 65 "	753 = 25 "	288 = 10 "	2 941	3 618	1.23	72	Gran
"	1 440 = 65 "	602 = 28 "	155 = 7 "	2 197	2 554	1.16	70	Aspl
"	763 = 74 "	164 = 15 "	117 = 11 "	1 044	942	.90	27	Woo
"	243 = 80 "	61 = 19 "	4 = 1 "	308	259	.84	7	Gran
New Orleans....	1 468 = 42 "	1 283 = 37 "	726 = 21 "	3 477	6 204	1.81	155	
"	842 = 76 "	277 = 23 "	15 = 1 "	1 134	1 065	.94	35	Aspl
Washington	3 133 = 70 "	1 196 = 26 "	191 = 4 "	4 520	4 622	1.02	66	
"	1 517 = 79 "	369 = 19 "	48 = 2 "	1 934	1 688	.87	34	
"	1 287 = 79 "	305 = 18 "	48 = 3 "	1 640	1 445	.88	29	Gran
"	902 = 71 "	307 = 25 "	56 = 4 "	1 265	1 289	1.01	21	Aspl
Buffalo.....	2 302 = 79 "	511 = 17 "	110 = 4 "	2 923	2 613	.83	47	Sand
"	386 = 47 "	217 = 26 "	219 = 27 "	832	1 505	1.88	36	Aspl
"	282 = 50 "	216 = 39 "	63 = 11 "	561	825	1.47	22	
"	372 = 65 "	144 = 25 "	60 = 10 "	576	714	1.24	14	Sand
Louisville.....	1 929 = 58 "	1 226 = 36 "	190 = 6 "	3 345	4 176	1.25	69	Gran
"	1 501 = 66 "	720 = 32 "	53 = 2 "	2 274	2 402	1.05	67	Aspl
"	627 = 68 "	272 = 29 "	30 = 3 "	929	977	1.05	28	Mac
Omaha.....	4 406 = 93 "	310 = 6 "	36 = 1 "	4 752	2 967	.62	47	Aspl
"	2 339 = 95 "	134 = 5 "	3 = 0 "	2 476	1 449	.59	24	Gran
Average.....	1889 = 67 per cent.	734 = 26 per cent.	216 = 7 per cent.	2839	3267	1.15	77	....

TABLE No. 1.

FIC RECORD ON CERTAIN STREETS IN VARIOUS AMERI

er 3 Tons.		Total.	Tonnage. Total.	Per Vehicle.	Per foot of width	Pavement.	Width in Feet.	
= 12	per cent.	7 811	10 905	1.39	273	Granite.....	40	Broadway,
= 2	"	5 460	3 744	.68	94	Asphalt.....	40	Fifth aven
= 4	"	2 357	2 357	1.00	87	Granite.....	27	Wall, cor.
= 18	"	6 081	9 237	1.52	142	Asphalt.....	65	Broad st.,
= 8	"	5 075	6 302	1.24	97	"	65	Filbert st.,
= 6	"	1 814	1 928	1.06	74	Granite.....	26	Chestnut,
= 23	"	3 637	7 561	2.08	151	"	50	Wabash, n
= 11	"	4 389	6 398	1.46	142	"	45	Clark, near
= 4	"	3 015	2 756	.90	77	Wood.....	36	La Salle, n
= 10	"	2 342	2 604	1.11	69	Asphalt.....	38	Dearborn,
= 2	"	5 362	5 301	.99	196	Wood.....	27	Devonshir
= 3	"	4 926	5 028	1.02	157	Granite.....	32	Devonshir
= 2	"	3 503	3 265	.93	126	Asphalt.....	26	Kilby, nea
= 0	"	3 591	2 938	.80	75	Granite.....	39	Washingto
= 1	"	1 484	1 130	.79	43	Asphalt.....	26	Arch, near
= 0	"	1 114	744	.67	32	"	24	Court squ
= 9	"	3 247	3 691	1.13	103	"	36	Locust, ne
= 10	"	2 941	3 618	1.23	72	Granite.....	50	Broadway,
= 7	"	2 197	2 554	1.16	70	Asphalt....	36	Pine, near
= 11	"	1 044	942	.90	27	Wood.....	36	Chestnut,
= 1	"	308	259	.84	7	Granite.....	36	Olive, nea
= 21	"	3 477	6 204	1.81	155	"	40	Tchoupito
= 1	"	1 134	1 065	.94	35	Asphalt.....	30	St. Charle
= 4	"	4 520	4 622	1.02	66	"	70	15th, opp.
= 2	"	1 934	1 688	.87	34	"	50	9th, bet. I
= 3	"	1 640	1 445	.88	29	Granite.....	50	7th, bet. I
= 4	"	1 265	1 289	1.01	21	Asphalt.....	60	6th, bet. I
= 4	"	2 923	2 613	.83	47	Sandstone..	56	Main, nea
= 27	"	832	1 505	1.88	36	Asphalt.....	42	Main, nea
= 11	"	561	825	1.47	22	"	38	Linwood,
= 10	"	576	714	1.24	14	Sandstone..	50	Main, nea
= 6	"	3 345	4 176	1.25	69	Granite.....	61	Main near
= 2	"	2 274	2 402	1.05	67	Asphalt.....	36	8th, near
= 3	"	929	977	1.05	28	Macadam...	35	7th, near
= 1	"	4 752	2 967	.62	47	Asphalt.....	63	Douglass,
= 0	"	2 476	1 449	.59	24	Granite.....	60	Farnham,
= 7	per cent.	2839	3267	1.15	77	.....	44	

	STREET.
0	Broadway, near Pine.
0	Fifth avenue, opp. Worth Monument.
7	Wall, cor. Broad,
5	Broad st., in front Pa. Railroad Depot.
5	Filbert st., in front City Hall.
6	Chestnut, corner 4th.
0	Wabash, near Lake.
5	Clark, near Madison.
6	La Salle, near Locust.
8	Dearborn, opp. Washington Park.
7	Devonshire, opp, Post Office.
2	Devonshire, near Milk.
6	Kilby, near State.
9	Washington.
6	Arch, near Summer.
4	Court square.
6	Locust, near Beaumont.
0	Broadway, near Olive.
6	Pine, near Garrison.
6	Chestnut, near Beaumont.
6	Olive, near Beaumont.
0	Tchoupitoulas, near Poydras.
0	St. Charles, near Washington.
70	15th, opp. Treasury.
50	9th, bet. D and E.
50	7th, bet. D and E.
50	6th, bet. Pa. ave. and B.
56	Main, near Swan.
12	Main, near Bouck ave.
38	Linwood, near Ferry.
50	Main, near Glenwood.
31	Main near 3d.
36	8th, near Walnut.
35	7th, near Jefferson.
33	Douglass, near 15th.
50	Farnham, near 14th.
44	

TABLE No. 2.

Traffic Record on Certain Streets in London and Liverpool.

City.	Vehicles.				Tonnage.					Street.
	Less than 1 Ton.	Between 1 Ton and 3 Tons.	Over 3 Tons.	Total.	Total.	Per Vehicle.	Per Foot Width.	Pavement.	Width in Feet.	
London	.....	.....	.....	12 148	13 507	1.11	422	Asphalt.	32	Gracechurch street.
"	.....	.....	.....	15 513	16 484	1.06	412	Wood.	40	King William street.
"	.....	.....	.....	8 167	8 330	1.02	378	Asphalt.	22	The Poultry.
"	.....	.....	.....	16 208	13 596	.84	367	Wood.	37	Strand and Fleet street.
"	.....	.....	.....	14 306	14 380	1.01	322	Macadam.	45	Parliament street.
"	.....	.....	.....	16 886	17 076	1.01	300	Wood.	57	Oxford street.
"	.....	.....	.....	9 419	9 260	.98	290	Asphalt.	32	Cheapside.
"	.....	.....	.....	6 128	7 588	1.08	253	"	30	Leadenhall street.
"	.....	.....	.....	10 776	9 358	.87	256	Macadam.	37	Piccadilly.
"	.....	.....	.....	12 132	10 658	.88	242	Granite.	44	Euston road.
"	.....	.....	.....	.....	.....	....	216	Wood.	..	Brompton road.
"	.....	.....	.....	6 371	6 506	1.02	203	Granite.	32	King William street.
"	.....	.....	.....	8 212	8 376	1.02	195	"	43	Edgware Road.
"	.....	.....	.....	10 796	9 668	.90	186	Macadam.	52	Regent street.
"	.....	.....	.....	.....	.....	....	156	Wood.	..	King's road.
"	.....	.....	.....	6 040	5 780	.96	145	Macadam.	40	Victoria street.
"	.....	.....	.....	.....	.....	....	93	Wood.	..	Sloane street.
Liverpool	.....	.....	.....	.....	.....	....	382	Granite.	..	(Street not named.)
"	.....	.....	.....	.....	.....	....	282	"	..	"
"	.....	.....	.....	.....	.. ...	....	231	Wood.	..	Great Howard street.
"	.....	.....	.....	.....	.....	....	100	"	..	Bold street.

Note.—The streets marked Macadam in the above list have all been repaved with wood since the date of Mr. Howarth's paper, owing to the expense of maintenance, which in some cases exceeded $1.00 per yard per annum. Edgware road and Euston road have also been paved with wood in place of the granite, on account of the noise arising from the latter.

The London records were not classified according to weight of vehicle, and hence I am unable to give the proportions of light, medium and heavy-weight vehicles. Colonel Haywood's report, however, classifies the vehicles according to the number of horses attached to them, and the proportion on King William street (wood) is 66 per cent. one horse, 31 per cent. two horses, and 2 per cent. three or more horses. The average of all the streets observed by him gave 71 per cent, 27 per cent. and 2 per cent. in the three classes. Mr. Howarth gives the number of cabs, omnibuses and loads above 3 tons on the streets named in his paper. On Piccadilly, for instance, with 10 776 vehicles:

6 084 (56 per cent.) are cabs averaging ½ ton each.

3 448 (32 per cent.) are omnibuses averaging 2 tons each.

728 (7 per cent.) are loads above 3 tons.

The great proportions of cabs and omnibuses explains at once the heavy traffic on London streets as compared with those of large American cities. In London no street railroads are allowed on any of the streets named in the above list, nor anywhere in the vicinity of them.

If there were street cars on these streets or in their vicinity, at least 80 per cent. of the omnibuses and 50 per cent. of the cabs would disappear.

This would take away 53 per cent. of the number of vehicles and about 70 per cent. of the tonnage. The tonnage on Piccadilly would be reduced to about 2 500 in place of 9 358, and 68 tons per foot of width in place of 253.

These figures show at once (what every one has noticed recently on Broadway) that street cars are a great relief to crowded streets, and that they effect a large saving in the wear of pavements. As laid in America, with T-rails and wooden sleepers, they are a nuisance to every other class of vehicle. As laid in Liverpool and London, with grooved rails and cast-iron sleepers bedded in concrete, they are almost unobjectionable. Vehicles pass over the rails without noticing them, and the only inconvenience arises from the fact that the cars cannot turn out in passing, and all other vehicles have to turn out for them.

In examining the figures of American traffic, we find that the average daily tonnage per foot of width is 77, and that it varies from 273 tons on Broadway to 7 tons on a granite street in St. Louis.

On some streets the traffic is remarkably uniform, and does not vary on any day more than 3 per cent. from the average; on other streets

the traffic is very irregular, being on some days as much as 50 per cent. greater or less than the average. In some cities bad weather makes a reduction in the volume of the traffic, and in other cities it does not. There seems to be no general rule for this. In regard to the hours of the day, the records everywhere show the same result—light traffic early in the morning and late in the afternoon; the maximum about 10 to 11 in the morning and 3 to 4 in the afternoon, and a decided falling off during the dinner hours, from 12 to 2.

As a general rule, the greatest traffic is on the business streets, but this is not always the case. In St. Louis, for example, Locust street is a purely residential street, leading from the business section towards the suburbs. Since it was paved with asphalt, its traffic has increased to 103 tons per foot of width, which is nearly 50 per cent. more than the traffic (72 tons) on the granite pavement of Broadway, the great business street of the town. Locust, Pine, Chestnut and Olive streets are all parallel and adjacent. There is nothing in their situation to indicate why the traffic should go to one of them more than to the others. They have been paved in the last few years—Locust and Pine, with asphalt; Olive, with granite; and Chestnut with wood on a concrete foundation. Their traffic is 103 tons on Locust, 70 on Pine, 27 on Chestnut, and 7 on Olive. In other words, the asphalt streets seem to have drained nearly all the traffic from the others. Similar results have taken place in Omaha. Farnham and Douglass streets were paved in 1882. The former being the main business street, it was thought necessary to pave it with granite, while Douglass street was paved with asphalt. The vehicles, however, choose the smooth pavement, and the traffic has been reversed, until now Douglass street has just twice as much as Farnham street. Similarly in Washington: Seventh street has been the principal business street for thirty years, and for that reason was paved with granite. Yet its traffic is now less than on the asphalt of Ninth street, which, when the two streets were first paved, was comparatively untraveled. Similar results have followed in Buffalo on the corresponding parts of Main street and Linwood avenue. It will be noticed that the wood pavement on Devonshire street, in Boston, supports a traffic of 196 tons—second only to Broadway.

The asphalt on Kilby street, in Boston, has a traffic of 126 tons, which is much larger than Washington street, in the same city (75 tons); Chestnut street, in Philadelphia (74 tons); Main street, in Buffalo (47

tons); Broadway, in St. Louis (72 tons); Main street in Louisville (69 tons); Seventh street, in Washington (29 tons); and Farnham street, in Omaha (24 tons), each of which is the main business street of its own city, and has been paved with granite under the supposition that its traffic was too heavy for asphalt.

The average tonnage per vehicle is an almost infallible indicator of the character of the street, *i. e.*, whether devoted to residential or business purposes. It ranges from .68 on Fifth avenue to 2.08 on a portion of Wabash avenue, in Chicago. The same character is indicated by the proportions of light and heavy vehicles on the street. On Fifth avenue, for instance, 91 per cent. of all the vehicles weigh less than 1 ton, while on Wabash avenue only 25 per cent. of them have so little weight. The general average is as follows: Less than 1 ton, 67 per cent.; between 1 and 3 tons, 26 per cent.; more than 3 tons, 7 per cent.; which agrees very closely with the proportions of one, two and three-horse vehicles observed in London by Colonel Haywood.

The table also shows that neither the maximum traffic nor the average traffic in each city, so far as here observed, is proportional to the population, although, as was to be expected, the largest cities have the largest traffic. In the five larger cities named in the above list, the average tonnage is 4 061, and 107 tons per foot of width; in the five smaller cities the average tonnage is 2 263, and 48 tons per foot of width. The streets in the larger cities have an average width of 37 feet, in the smaller cities 49 feet. The importance of laying out streets of proper width is clearly shown. Comparing, for instance, Wabash avenue with Devonshire street in Boston, we find the total tonnage on the Chicago street is 7 561 tons, while in Boston it is 5 362; yet the Boston street has to support 196 tons per foot of width, against only 151 tons in Chicago, because the width of the former is only 27 feet, while the latter has 50 feet.

The average tonnage per foot in each city, so far as here observed, varies from 151 in New York to 30 in Buffalo, and the general average is 77. A great many more observations would have to be taken, however, before attaching much importance to these averages. Mr. Deacon, in discussing the cost of maintaining various pavements, reduces the cost in each case to a standard traffic of 100 000 tons per yard per annum, which is very nearly the same as 100 tons per foot per diem. Mr. Stayton uses 150 tons per diem as a standard of average traffic. Both of these

writers assume that the wear of the pavement is a direct function of its wheel-tonnage per unit of width. It is doubtful if the problem is so simple as this; there are other factors, such as horse-tonnage, grade, climate, proportion of light and heavy vehicles, etc. But it is certainly true that the wheel-tonnage is the most important factor, and is almost the only one which can be readily stated in figures. Hence it is approximately accurate to assume that a pavement of the same materials, laid in the same manner, should wear twice as rapidly under twice the tonnage per unit of width. With this assumption, both Mr. Deacon and Mr. Stayton compare the maintenance cost of various pavements reduced to the same traffic standard, *i. e.*, the standard being 100 tons, if the pavement has 200 tons traffic per foot, its maintenance cost is divided by ½, and if it has only 50 tons, it is multiplied by 2 before comparing them, in order to see which is essentially the cheapest and most durable pavement.

This is, of course, the practical object of recording the traffic on the street. The first cost of a pavement is only one of the factors which determine its cost in a number of years. To determine this it is necessary to take account of the cost of maintenance and the interest on first cost. And in order to determine the inherent merits of the pavement, the volume of traffic which causes the expense of maintenance should also be taken into account, as has been done by the English engineers above mentioned. It is needless to say that there are no statistics of the cost of maintenance in this country that would enable any such comparisons to be made. Any one who doubts the utility of such records and comparisons has only to contrast the miserable condition of the vast majority of American streets as compared with those of England, where the paving question has been scientifically studied by means of such records, in order to have his doubts removed.

In compiling the reports of accidents, I have followed the method adopted by Colonel Haywood, *i. e.*, the number of horses is multiplied by the distance over which they are observed, in order to obtain the total distance traveled. This is divided by the total number of accidents in each class in order to find the distance traveled before an accident occurs. This is the standard of comparison. The accidents are divided into three classes, viz.: Falls on knees, falls on haunches and complete falls. The number of horses was not actually counted, but was obtained by estimating the light-weight vehicles to have one

horse, and the medium and heavy weight two horses each. This gives a result which is probably below the actual number of horses.

The result of the observations compiled in the manner indicated, is given in Table No. 3.

The general result of these observations—which it should be noticed were made on 192 days, in thirty-two different streets of ten different cities, and include 807 552 horses and 81 051 miles traveled—is as follows, viz., a horse will travel the following distance before meeting with an accident of any kind:

On asphalt	583	miles.
On granite	413	"
On wood	272	"

Colonel Haywood's results were as follows:

On asphalt	191	miles.
On granite	132	"
On wood	330	"

The result of both sets of observations thus accords in showing that there are fewer accidents on asphalt than on granite. They do not accord in regard to wood, the London experiments showing it to be the least slippery and ours the most slippery of all the three classes. On account of the small number of our observations on wood, it is hardly fair to make the comparison, but there are very few cities in which wood is now used in America, and hence a greater number of observations could not be obtained. In the above statement it will be seen that there are about three times as many accidents on the London pavements as on the American. This is due partly to the dampness and fogs of London and partly to the greater traffic, and consequently greater amount of manure. The manure and fog combined make up the "grease" from which the London streets are seldom free, although their system of street cleaning is, I think, the best in the world. The fine grained Neuchatel asphalt, which is used in London, is also much more slippery than the gritty Trinidad asphalt used in this country.

In further classifying the accidents, we obtain the Table No. 4:

TABLE No. 3.

GREENE ON STREET TRAFFIC.—ACCIDENTS TO HORSES. 1.—

	Horses.				Ac	
	Number Observed.	Average Load.	Average Distance in Miles.	Total Distance Traveled in Miles.	Falls on Knees.	Falls on H'nche
New York	35 778	.62	.028	1 002	2	1
Philadelphia	55 248	1.03	.133	7 348	0	9
"	41 940	.91	.176	7 380	0	0
Chicago	17 922	.80	.113	2 025	0	0
Boston	26 562	.76	.077	2 045	10	5
"	10 344	.66	.078	807	1	1
"	4 940	.60	.047	235	0	1
St. Louis	25 416	.89	.113	2 872	6	0
"	17 724	.84	.113	2 003	0	0
New Orleans	8 646	.73	.113	977	1	1
Washington	35 442	.78	.076	2 701	6	0
"	14 106	.71	.076	1 072	1	1
"	9 768	.80	.038	371	0	0
Buffalo	7 548	1.20	.680	5 131	0	0
Louisville	18 282	.80	.113	2 066	0	0
Omaha	30 588	.58	.119	3 392	0	0
Total	360 254			41 427	27	19
Distance traveled before an accident occurs. Miles					1 534	2 180

2.—GRANITE PAVEMENTS.

New York	67 458	.97	.057	3 846	14	2
"	18 055	.78	.057	1 029	7	1
Philadelphia	14 488	.80	.113	1 590	0	0
Chicago	38 205	1.19	.113	4 317	0	0
"	39 084	.95	.113	4 416	0	0
Boston	38 754	.78	.071	2 748	8	0
"	26 006	.66	.064	1 664	10	1
St. Louis	23 952	.90	.113	2 716	8	0
New Orleans	32 916	1.13	.057	3 990	17	1
Washington	11 958	.72	.076	909	4	1
Buffalo	21 264	.73	.113	2 403	0	0
Louisville	28 566	.88	.113	3 230	0	0
Omaha	15 678	.55	.119	1 865	0	0
Total	376 384			34 723	68	6
Distance traveled before an accident occurs. Miles					510	5 954

3.—WOOD PAVEMENTS.

Chicago	21 972	.75	.113	2 483	0	0
Boston	41 700	.76	.045	1 876	10	4
St. Louis	7 242	.78	.075	542	2	1
Total	70 914			4 901	12	5
Distance traveled before an accident occurs. Miles					408	980

TABLE No. 3.

ON STREET TRAFFIC.—ACCIDENTS TO HORSES. 1.—ASPHALT P

	Horses.				Accidents.		
	Number Observed.	Average Load.	Average Distance in Miles.	Total Distance Traveled in Miles.	Falls on Knees.	Falls on H'nches.	Complete Falls.
.....	35 778	.62	.028	1 002	2	1	2
.....	55 248	1.03	.133	7 348	0	9	10
.....	41 940	.91	.176	7 380	0	0	0
.....	17 922	.80	.113	2 025	0	0	0
.....	26 562	.76	.077	2 045	10	5	4
.....	10 344	.66	.078	807	1	1	2
.....	4 940	.60	.047	235	0	1	1
.....	25 416	.89	.113	2 872	6	0	0
.....	17 724	.84	.113	2 003	0	0	1
.....	8 646	.73	.113	977	1	1	1
.....	35 442	.78	.076	2 701	6	0	3
.....	14 106	.71	.076	1 072	1	1	0
.....	9 768	.80	.038	371	0	0	0
.....	7 548	1.20	.680	5 131	0	0	0
.....	18 282	.80	.113	2 066	0	0	1
.....	30 588	.58	.119	3 392	0	0	0
.....	360 254	.....	.....	41 427	27	19	25
t oc-							
.....	.....	.....	.....	.....	1 534	2 180	1 647

2.—GRANITE PAVEMENTS.

.....	67 458	.97	.057	3 846	14	2	0
.....	18 055	.78	.057	1 029	7	1	1
.....	14 488	.80	.113	1 599	0	0	1
.....	38 205	1.19	.113	4 317	0	0	0
.....	39 084	.95	.113	4 416	0	0	1
.....	38 754	.78	.071	2 748	8	0	0
.....	26 006	.66	.064	1 664	10	1	0
.....	23 952	.90	.113	2 716	8	0	2
.....	32 916	1.13	.057	3 990	17	1	4
.....	11 958	.72	.076	909	4	1	0
.....	21 264	.73	.113	2 403	0	0	0
.....	28 566	.88	.113	3 230	0	0	1
.....	15 678	.55	.119	1 865	0	0	0
.....	376 384	.....	.....	34 723	68	6	10
t oc-							
.....	.....	.....	.....	.....	510	5 954	3 472

3.—WOOD PAVEMENTS.

.....	21 972	.75	.113	2 483	0	0	0
.....	41 700	.76	.045	1 876	10	4	0
.....	7 242	.78	.075	542	2	1	1
.....	70 914	.....	.....	4 901	12	5	1
t oc-							
.....	.....	.....	.....	.....	408	980	4901

HALT PAVEMENTS.

NTS.		STREET.
omplete Falls.	Total.	
2	5	Fifth avenue.
10	19	Broad street.
0	6	Filbert street.
0	0	Dearborn street.
4	19	Kilby street.
2	4	Arch street.
1	2	Court square.
0	6	Locust street.
1	1	Pine street.
1	3	St. Charles street.
3	9	Fifteenth street.
0	2	Ninth street.
0	0	Sixth street.
0	0	Main street.
1	1	Eighth street.
0	0	Douglass street.
25	71	
1 647	583	
0	16	Broadway.
1	9	Wall street.
1	1	Chestnut street.
0	0	Wabash avenue.
1	1	Clark street.
0	8	Devonshire street.
0	11	Washington street.
2	10	Broadway.
4	22	Tchoupitoulas street.
0	5	Seventh street.
0	0	Main street.
1	1	"
0	0	Farnham street.
10	84	
3 472	413	
0	0	La Salle street.
0	14	Devonshire street.
1	4	Chestnut street.
1	18	
4901	272	

TABLE No. 4.—Distance Traveled (in miles) before an Accident Occurs:

	Falls on Knees.	Falls on Haunches.	Complete Falls.	Accidents of any kind.
Asphalt	1 534	2 180	1 647	583
Granite	510	5 954	3 472	413
Wood	408	983	4 901	272

From which it appears that in falls on knees the order of excellence is asphalt, granite, wood; in falls on haunches, granite, asphalt, wood; and complete falls wood, granite, asphalt. This shows that while a horse will bark his knees more frequently on granite than on asphalt, yet he will be thrown, by pulling him up suddenly and throwing his hind legs from under him, more frequently on asphalt than on granite. Nearly every "complete fall" on asphalt is reported as due to this carelessness in driving. On the other hand the accidents on granite are largely due to bad blocks. Even in the newer granite block pavements the mistake is made of using blocks nearly five inches in width. If the horse misses his hold on the first joint his foot is pretty sure to go out from under him. In Manchester and Liverpool, whose granite pavements are the admiration of every one who has seen them, the blocks average only three inches in width; and the horse has a chance to catch on the second joint if he slips from the first.

For the purpose of comparison, the Table No. 5 is given, showing the percentage of each class of falls, as taken from Colonel Haywood's report, and from my own observations:

TABLE No. 5.

	London.			American Cities.		
	Falls on Knees.	Falls on Haunches.	Complete Falls.	Falls on Knees.	Falls on Haunches.	Complete Falls.
Asphalt	32 per cent.	24 per cent.	44 per cent.	38 per cent.	27 per cent.	35 per cent.
Granite	46 "	8 "	46 "	81 "	7 "	12 "
Wood	85 "	3 "	12 "	66 "	28 "	6 "

The principal difference in these two tables is the much greater proportion of complete falls on the granite pavements of London in comparison with those of America. The reason of this is not clear. It may be due to the fact that a horse who loses his fore-feet on the London streets is apt to go down altogether on account of the "grease," whereas he may recover himself on the American streets.

Colonel Haywood's report classifies his accidents according to grade of street, cleanliness of street, state of the weather, rate of travel and method of harnessing. Into these refinements I have not attempted to enter. The only classification I have made is according to the nature of the pavement. With one exception the grades here observed were all flat, *i. e.*, less than 2½ in 100, and no distinction was made for weather or cleanliness.

The general result of the observations, both in London and in this country, shows that there is much less difference as regards slipperiness between asphalt and granite pavements than is generally supposed; that for falls on the knees granite is more slippery than asphalt, while for falls on the haunches asphalt is more slippery than granite; and that, without classifying the nature of the fall, about 40 per cent. more horses fall on granite than on asphalt. It is needless to remark that these facts are quite opposed to the popular opinion on the subject.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

322.

(Vol. XV.—February, 1886.)

REMARKS ON THE AERATION OF WATER.

BY CHARLES B. BRUSH, M. Am. Soc. C. E.

READ AT THE ANNUAL CONVENTION, JUNE 26TH, 1885.

The following statement relates to the process for the aeration of water, as introduced during the past year on the works of the Hackensack Water Company, and as now being introduced in the City of Philadelphia. The Hackensack Water Company, reorganized, supplies that portion of New Jersey on the west bank of the Hudson River, lying opposite that portion of the City of New York above Grand street, and including Hoboken, Union Hill, Weehawken, North Bergen, Ridgefield, Hackensack and other adjacent places. The water is taken from the Hackensack River at New Milford, about five miles above Hackensack; from thence it is pumped 14 miles through a cast-iron main to the principal reservoir on the heights north of Hoboken. The Hackensack Reservoir is connected with this main about three miles from the pumping station.

During June, 1884, an unpleasant taste and smell was first noticed in the water furnished to Hoboken; in July these peculiarities became

very pronounced, and then a green scum began to collect on the water in the Hoboken Reservoir. After a while this took the appearance of green paint. The reservoirs were full in anticipation of the summer drought. There was no unpleasant taste or smell from the water drawn along the force main; none from the water in the river; none in the water supplied to Hackensack; and none in the Hackensack Reservoir; but after the water was delivered into the Hoboken Reservoir the taste and smell became offensive. As soon as this green scum appeared on the surface of the water in the Hoboken Reservoir, the water in that reservoir was cut off from the city, and Hoboken was supplied directly from the pumps. I found that by keeping the water in motion from the time it left the river until it was delivered to consumers in Hoboken, the unpleasant taste and smell largely disappeared. The assistance of Dr. Leeds, of Stevens Institute, was then obtained. We found that this green scum continued to increase on the surface of the water in the reservoir until a white scum appeared in the middle of it, with a further development of bluish streaks and a foam-like appearance, culminating finally in white patches over much of the bluish-green surface. This remarkable appearance was confined to the surface, the water 2 feet below not being visibly affected. When the wind sprang up this discoloration disappeared. Analyses, frequently made, showed that there was a deficient supply of oxygen in the water, and that this development of green *algæ* increased as the supply of oxygen in solution in the water decreased. In its normal condition the amount of oxygen in solution in good water is about 6½ cubic centimeters per liter, $\frac{6.5}{100}$ of 1 per cent. by volume, but in this case it had run down to about 3½ cubic centimeters. The drainage area of the works is about 100 square miles. There is no sewage pollution in this area. The difficulty was entirely of vegetable origin. Since the deficiency in oxygen, together with a somewhat large percentage of dissolved extractive matters of vegetable origin, were the only abnormal features revealed by chemical analysis, Dr. Leeds suggested that we could improve the water by supplying the oxygen requisite to bring the water to its normal condition, and probably succeed in oxidizing the dissolved extractive matters at the same time. He had found by laboratory experiment that he could cause the offensive taste and smell which affected the Philadelphia water supply as taken from the Schuylkill River in January, 1883, to entirely disappear, and had been led to devise a process by

which aeration could be easily applied. The benefit of aeration of potable water has indeed been recognized from time immemorial, and had already been made the subject of certain patents in this country by Mr. R. d'Heureuse, but these involved the use of air at merely ordinary atmospheric pressure. Dr. Leeds controls these patents, but has improved upon them by introducing the air under greater pressure, which not only causes the work of oxidation to be very rapidly and effectually performed, but makes the process of such a character as to be easily applied in practice. Under his advice we set up air compressors at New Milford, and forced air into the mains under a pressure of about 125 pounds to the square inch. By so doing the oxygen in the water is increased, and, ordinarily, when the water is not turbid from suspended earthy matter (a difficulty encountered after heavy storms, and which, of course, can only be completely removed by filtration), it manifests a sparkling appearance and has only a pleasant taste and smell. The water as drawn from the main is often perfectly white, but in a moment it clears up from the bottom like soda water, and those who take the water directly from the main drink it with delight while still effervescent. We commenced in September, 1884, with this aeration process. We had no difficulty during the fall and expected none; but during the winter there was a taste and smell in the water which was supplied to Hackensack. There was some 40 or 50 days' supply in the reservoir of that town, which was covered with ice 2 or 3 feet thick, and all the streams that supplied our pumping station were covered with ice. In order to test the aeration theory, we pumped air directly into the distribution main at Hackensack; within six hours the smell and taste had disappeared from the water in these mains. Then we cut the ice and pumped air into the water in the reservoir, after which the difficulty there also disappeared. On May 30th, 1885, the smell and taste again appeared in the water supplied to Hoboken. We were partially aerating the water at the time; we increased the amount of air supplied and the smell and taste entirely disappeared. At present the condition of the water seems to be excellent. We are, it is true, just entering our dangerous season. Perhaps the aeration process will not carry us through, but we believe it will. We are careful to keep the water in our reservoirs so moving that every part shall be supplied with aerated water. We also keep the reservoirs properly stocked with fish, principally carp and black bass, removing them as they become too numerous. Analyses of the water from different points

are made once a month, and sometimes oftener. Microscopic examinations are made also, which show that the animal life is changed in different conditions of the water.

DISCUSSION.

Col. WILLIAM E. MERRILL, M. Am. Soc. C. E.—Drinking water is presumably affected by sunlight. I observed some years ago that all the reservoirs in Paris were entirely covered, so that the water was kept in the same condition as it would be in any subterranean reservoir, and it occurred to me that many of these vegetable organisms that appear on water and cause this taste and smell are due to the vivifying action of the sun. I would like to ask whether the supply to Hackensack and Hoboken is exposed to the sunlight.

Mr. BRUSH.—Most of it is exposed to the sunlight. The flow of the river is from 20 000 000 to 600 000 000 gallons per day, and the worst stage is at its time of least flow. I believe that the offensive condition of the water is due to the slow flow and the decomposition of vegetation in the river. There is not vitality enough in the water to resist, burn up, or throw off the impregnating effect of decomposing vegetation. As long as the water is actually in motion it does not stagnate and ferment; but will do so soon after coming to rest. If I am right in this view, I do not see that any practical gain will be obtained by covering the reservoir; this operation will always be expensive, and often impracticable. If the water contains sufficient oxygen in solution, I believe that the more sun, air and motion we give to water the better it will be.

Col. MERRILL.—That is so contrary to European practice, that I think it ought to be discussed.

J. F. FLAGG, M. Am. Soc. C. E.—This scum is vegetable matter?

Mr. BRUSH.—It is the result of vegetation.

Mr. FLAGG.—Is it growing vegetable matter?

Mr. BRUSH.—No, it is not growing. It looks like green paint.

Mr. FLAGG.—Do you suppose it is decomposed?

Mr. BRUSH.—Yes, in some process of decomposition. I take it that the wild plants along the river would naturally improve the water, yet when the water gets to that slow stage of flow, vegetable life appears which is injurious. This vegetation does not appear while the water is kept in active motion.

JAMES B. FRANCIS, M. Am. Soc. C. E.—What is the depth of the reservoir ?

Mr. BRUSH.—The reservoir is not large. The main reservoir is 16 feet deep; the small one 13 feet.

Mr. FRANCIS.—Is there any marked difference in the water as you go down ?

Mr. BRUSH.—I do not allow much difference to appear, because I keep it in constant motion. After that trouble at the Hackensack River I found that the reservoir at Hackensack, which was built long ago, before I had anything to do with it, was built in such a way that the lower eight feet had no circulation and the water was taken directly above that, and my impression was that the taste and smell came from that lower portion. I drew that all off and there has been no complaint received since.

Mr. FRANCIS.—I would like to say to Col. Merrill that the covering of the reservoirs in Europe has a different object in view from the one he mentioned, namely, to protect the water from dust and soot. Many of them are near cities where dust and soot accumulate very rapidly.

Col. MERRILL.—That no doubt is a very important object. But is it possible for any organism to grow in water unless you have some ozone ? In an artesian well you have pure water—nothing can happen to it; no vegetable organism can appear. Now, if you put water where no sun can get at it, is it not likely to be pure ? Can anything live without sunlight ? It seems to me that if Mr. Brush would combine his method of aeration with this method, keeping the water away from the influences that cause life, then the combination would be perfect.

J. N. TUBBS, M. Am. Soc. C. E.—I agree to a certain extent with Colonel Merrill. I am connected with water-works in Rochester, N. Y. Of course these odors in water are produced by a variety of causes. Our experience was something like this. Our water supply is taken from a mountain lake about seven miles long and one-half to three-quarters of a mile wide. It is carried in a close conduit of iron and discharged into a reservoir, and then carried nine miles and discharged into another reservoir. In 1876, the first year the works were opened, a very disagreeable smell, denominated "fishy," was discovered in the water, and it continued during the months of September and October, and even later. The conduit then extended out into the lake itself about 600 feet by a flexible-joint pipe three feet in diameter and opening three or

four feet from the surface. The trouble continued for two or three months. In the succeeding year this pipe was put into thirty feet of water; in the year after that some difficulty occurred at the Lake. I happened to be there, and it was evidently from vegetable growth that had reached maturity at the bottom and came to the surface of the Lake, perhaps by the propeller of a steamboat stirring it up. It had an appearance like fine brick-dust. There was no particular odor or taste to the water until the July sun came upon it, when the vegetable organisms seemed to decay, and it was the decay of these which produced the bad smell. This did not go down any great depth—two and a half feet, perhaps. All around the shore of the lake was that smell like decaying fish, but none of this reached the depth of thirty feet where the pipes were. In the construction of the works we had arranged to throw the whole supply of water into the air for the purpose of aeration. A twenty-four inch main is carried up through masonry and is discharged in twenty-one jets into the air under the head of the storage reservoir eight or nine miles away. It goes up into the air and is thoroughly aerated. This was done while the fishy odor was present, but while it did some good it did not seem to do much. I am of opinion that these vegetable organisms grow in patches in localities beneath the water, and when they reach the surface the sun causes them to die and then there is a disagreeable smell. I think that when this occurs no aeration will do much good, and the remedies must be applied to the source of the growth of these organisms; and I believe, further, that the suggestion of Col. Merrill, that the covering over of the water, without preventing the free access of the air, will, to a large extent prevent the decay and the disagreeable results following.

J. J. R. Croes, M. Am. Soc. C. E.—In some places where ground water is depended upon for the supply of water to towns—and is drawn from covered galleries, there has not, so far as I know, been trouble from the growth of *algæ;* but in a number of cases where pure spring water is received in open galleries, on exposure to the sunlight in the basins where it is received, nearly every summer the water becomes intensely disagreeable, getting a cucumber or a fishy taste. That has been the case in Taunton, Mass., where an open gallery was used. At certain periods of the year there has been trouble from the fishy taste and smell. During last winter that gallery was extended and a connection made with the river; not a pipe directly, but on the

river-side an embankment of coarse gravel and broken stone was built, through which the water percolated, filtering it to a certain extent. Just before the river water was introduced into the gallery, this smell and taste occurred in the old basin, but in a letter from the Superintendent, Mr. Billings, he said that since the introduction of the river water into the basins, part of the supply being taken from the ground water and part from the river, the taste and smell had entirely disappeared; that the mixture of the two waters had caused the vegetable organisms to disappear. I think that in no case where these underground basins are entirely covered, and the water protected from the air until it has reached the consumers, has there been any unpleasant odor or taste; but in every case where this underground water is exposed to the sunlight before it is pumped into the mains, something of the kind has occurred during the summer.

Mr. J. B. Francis.—The experience in Boston in the aqueduct might favor Col. Merrill's idea. Vegetable matter upon the aqueduct there, when examined, was found generally to be very dense at the commencement of the aqueduct, gradually declining until it disappeared at some distance. Whether that was caused by getting into the dark is a question.

Mr. Brush.—The water we supply to our high-service consumers on Union Hill was not exposed to the sun. It was taken directly from the main and stored in a covered tank open to the air. During our trouble the complaint from our high-service consumers was as emphatic as from those who were supplied directly from the Hoboken Reservoir.

Emil Kuichling, M. Am. Soc. C. E.—I have seen quite a number of these covered reservoirs and I have not been able to find that even when covered completely they serve to prevent the growth of vegetable matter. A case in point is one at Hanover, a new reservoir. There was vegetable matter occurring there which I saw myself, a fine species of *algæ* without much color. Biological researches have shown that the color of these vegetable organisms is dependent upon the amount of light they receive. Some species of *algæ* were cultivated in the laboratory at Leipsic in a glass tube, and as they exposed this glass to the light, it developed according to the intensity of the light a red color or a green color upon the *algæ*. In some cases they were simply colorless and yet they could be detected. In Berlin they have a series of wells along the shore of Tegeler Lake, and in these

wells they discovered a few years ago a growth of vegetable matter which has been the subject of investigation by some of the most eminent scientists of Berlin. It grows in the filtering gallery and also in their force mains to such an extent as to almost close up the mains. The exclusion of light is therefore not sufficient to produce immunity from this vegetable growth. A number of other wells in Europe as well as in this country show growths of *algæ*. There is not so much smell or taste in waters thus stored, which is due perhaps to the fact that they are protected somewhat from the sun and from the effect of sunlight itself. The cases observed indicated a slight taste of the water, also a slight smell, but not sufficient to become offensive. Heat and light together will cause a different state of growth and development of these plants than if they were placed in abnormal conditions of darkness. The spores will get into water. No amount of filtration has sufficed to keep these germs out. The experiments of Professor Pumpelly for the National Board of Health two or three years ago, showed that these germs could still be cultivated and were in water after filtration through one hundred feet of sand. Now if we find these spores of vegetable matter in a well sunk into the subsoil of broad valleys, and also find them after filtration, in storage reservoirs, it seems to me that the question is still a debatable and an unsolved one how to treat them. The aeration of the water at Rochester, as passed up though the fountain of the distributing reservoir, had the effect of modifying the taste to a large extent, but standing to the leeward side of this fountain one could easily detect, three hundred feet from it, the peculiar fishy smell, although there was a considerable improvement in the quality of the water.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

323.

(Vol. XV.—March, 1886.)

THE ABT SYSTEM OF RAILWAY FOR STEEP INCLINES.

By WALTON W. EVANS, M. Am. Soc. C. E.

READ OCTOBER 21ST, 1885.

The problem of how to climb steep inclines on railways in the best and most economical way, has taxed the inventive powers and exercised the brains of many and clever engineers. A third of a century or more ago, almost all of the important valleys of the civilized world had been occupied by railways depending on weight and the adhesion of smooth surfaces as the chief elements of traction in moving railway trains. As the engine began to mount inclines, and as there was an ever-increasing desire to haul heavier trains, the locomotive was enlarged in all its capacities and made more ponderous. The rail had to grow in weight and size to be able to carry the locomotive and perform its part of the duty, and excellent as have been the results on easier gradients in the reduction of cost of transportation, when we come to steeper inclines the service fails in an alarmingly rapid degree. There the locomotive with its five pairs of wheels coupled and a total weight of 144 000 pounds, having 128 000 pounds on the driving-wheels, has become a perfect giant. Yet a giant who is chiefly seen wrestling with his own stupendous mass is a sore exhibit of impotence, and this brings us to a point where we may with propriety say, stop, we can go no further, we have come to

conditions where these Saturn-like monsters are devouring everything but the empty pockets of the shareholder. We know that there are rich valleys up in mountain districts; we know that there are rich veins of minerals up in mountain regions that the greed of man and his ever-restless march to the front is anxious to develop. There is a trite old saying that "Where there's a will, there's a way." It would be a sad thing if we had to cry "halt" in railway progress, and had arrived at a point beyond which we could not go. We should hail with joy any invention that bids fair to let us take another step in advance in solving this great problem of making the steel-shod horse ascend the inclines of a mountain side.

The writer has studied the inventions and improvements of Mr. Roman Abt, of Switzerland, and believes that he has done a great deal towards solving the problem we are interested in, and by so doing he has opened new fields for railway enterprise and for the athletic exercises of the iron horse. But before entering on a description of the Abt system of railway, and the machinery he uses to produce economical results, it is proper to review the past history of the designs and inventions connected with overcoming steep railway gradients. It is always fascinating to note how ideas spring up, prevail for a time, are finally rejected, to be taken up anew and stamped in the molds of advanced and progressive arts. Everything has its proper time. Slow is the course of true progress, but, "Chi va piano, va sano."

Historical Review.

When Trevithick, in 1804, had succeeded in building the first locomotive engine that ever pulled a train of loaded cars, an event that took place on the Merthyr Tydvil Railway in South Wales, he relied merely on the adhesion between his smooth driving-wheels and smooth rails for obtaining the requisite traction. His locomotive being light, the tractive power appeared to be very small, and yet the engine weight was more than the cast-iron tram-plates of those days could stand. Instead of grasping the proper remedies of greater engine weight combined with stronger rails—those remedies which led to the wonderful developments of later years—Trevithick, partly also because called away by other interests, gave up all further attempts to improve upon his locomotive, and expressed the conviction that nothing but some additional means of propulsion could yield satisfactory results.

A suggestion like this, coming from so prominent an engineer as Trevithick, did not fail to shape the course of inventive genius for the following years. But nothing of a practical nature was developed until, in 1811, Blenkinsop obtained an English patent for the application of a rack laid all along the line of a railway, with a corresponding driving-pinion on the locomotive. This plan was at once executed on a railway leading from Leeds, in England to the Middleton Coal Pits. The opening of the line occurred August 12th, 1812, and though rack and engine were of an extremely primitive construction, yet this mode of operation remained in use for over twenty years in the transportation of coals from the Middleton Pits to Leeds. The pinion alone furnished the traction, and the engine worked on gradients of 1 in 15. Here, then, appears for the first time an effective means for operating heavy inclines by locomotive power. The action of the pinion in the rack is positive, and the amount of traction is limited only by the dimensions of rack and pinion, and is practically independent of the engine weight.

Fruitful as this idea would have seemed to be, and notwithstanding the fact that the light locomotives of the earlier railroad era excluded any economic operation of steeper gradients, the rack and pinion found no friends in its immediate adaptation to the regular railway system. The lines were universally developed with easy gradients—even Stephenson himself thought that the economical limit was reached with a gradient of 1 in 200, a limit which De Pambour later on extended to 1 in 100, and whenever a sudden rise or steeper gradients were encountered, they were overcome by means of ropes and stationary engines, much in the fashion that inclined planes in coal pits had already been worked for a long time. These inclined rope sections once formed a prominent feature on many an important railway in Europe, as well as in the United States; but one by one they were adapted to regular locomotive service, and with perhaps some few exceptions, on roads of secondary importance, they have long since become a mere matter of history. Inconvenience and expense have sealed their fates.

The so-called atmospheric railways formed a curious phase in the struggle for supremacy among the means of transportation. They took practical shape during the interval from 1840 to 1848, and occupied the closest attention of prominent engineers. Millions were spent in England and France in the perfection of these schemes, and millions

and schemes disappeared together. It had been expected that this mode of propulsion would be of special use in the operation of steep inclines, and it therefore deserves a passing mention.

In 1847, a bold attempt was made in our country to adapt the rack and pinion to the requirements of a regular railway line. The experiment took place on an inclined plane of 1 in 17 of the Madison and Indianapolis Railroad.* Besides the regular adhesion devices, the locomotive was provided with an independent and separately working steam engine for driving the pinion mechanism. Both adhesion and pinion worked simultaneously and assisted each other, and this co-operation constituted a signal advance over the original arrangement of Blenkinsop. But there was no end of repairs to the cast-iron rack, and these expensive repairs, and a somewhat defective design of the engines, led first to rope traction as a substitute, which, in the natural course of things, as elsewhere, gave way to locomotive service. As long as a cast-iron rack-rail had to be employed, this system had no possible chance for any larger applications, and with the state of the arts in those days, it was bound to fail. Everywhere adhesion was made to do the service as best it could.

Almost simultaneously with this described effort on the Madison and Indianapolis Railroad, George Escol Sellers, of Cincinnati, appeared with a plan for working inclines by laying a smooth center-rail, and by supplying the regular locomotive with a pair of horizontal friction rollers, pressing againt the center-rail, and being operated by a special set of cylinders. These friction rollers were to produce an additional tractive power sufficient to overcome very steep inclines. The proposition of a center-rail and horizontal friction wheels as a means to increase traction had already been made by Vignolles and Ericsson, previous to the historic competition on the Liverpool and Manchester Railway, but George Escol Sellers appears to have been the first one to distinctly propose the use of a smooth center-rail for climbing steep gradients. His plan, as laid down in a comprehensive and very lucid pamphlet, published in Cincinnati in 1849, contains many very ingenious suggestions and details, among which may be mentioned the clever manner by which

* A full account of the inclined plane near Indianapolis, with descriptions of the rack-rail, and of various locomotives and attachments used in connection with it, and also of the final use of the same plane by a gravity locomotive, is given in paper No. 158, by M.J. Becker, M. Am. Soc. C. E., in the Transactions of the American Society of Civil Engineers, Vol. VI, page 68 (March, 1878).

the resistance of the train was made to create the requisite pressure of the friction wheels against the center-rail. Mr. Sellers had a working model built, which was first exhibited in Cincinnati and subsequently in New York in 1848. When, stimulated by the all-exciting gold discoveries in California, the construction of the Panama Railway was decided upon, the first surveys led to gradients for which the ordinary means of locomotion appeared to be insufficient. This gave a chance for a practical demonstration of Mr. Sellers' plan on a grand scale, and the first engines for the Panama Railway were actually ordered and built upon his system. In the meantime, however, and before these locomotives arrived at the Isthmus, the present line, with comparatively light gradients, had been discovered. The horizontal adhesion mechanism became superfluous, and the engines were worked without it.

The year 1851 was one of special interest in the eventful development of the iron horse. It was the year in which the great competitive trial took place on the Semmering Railway in the Noric Alps of Austria, which was to decide on the best type of adhesion engine to work that difficult line, with its extensive gradients of 1 in 45, 1 in 42, 1 in 40, and with its numerous curves. None of the competing locomotives answered satisfactorily in the subsequent regular service, and this was productive for several years of many projects in locomotive construction for working steep gradients. Among these we notice the first appearance of the *double bogie engine* by Cockerill, in Seraing, Belgium, which, in very recent years, flashed up again into an ephemeral existence. We also recognize here Sellers' center-rail and mechanism in a proposition by Krauss, but all these plans found no favor, and had to give way to the *Engerth type* of locomotives, and its gradual transformation into the improved eight-couplers of our present day.

Once more we meet the center-rail of Sellers, when Fell, in 1867, applied it practically in his railway across the Mont Cenis. That line had an average gradient of 1 in 25.6, with maximum gradients of 1 in 12. It remained in actual operation until the tunnel was opened to traffic. This formed the last practical attempt with a smooth center-rail. The engine was rather complicated. The tractive power depended entirely upon the adhesion of smooth surfaces, and varied thus with the state of the weather and the conditions of the track.

But while Fell was busily engaged laying his center-rail across the Alps, Sylvester Marsh, in this country, was at work running a line with

a center rack-rail to the top of Mount Washington.* There could be no doubt as to the steepness of this road, its average gradient being about 1 in 4.1, with a maximum gradient of 1 in 2.67. The ground was first broken in the summer of 1866. With this Mount Washington Railway, the rack-rail system of Blenkinsop entered into a new era of development, and this was essentially due to the improvements in the rack itself. Cast-iron, with its low limit of strength and its unreliable qualities, was abandoned, and wrought-iron was used in its stead for the first time. The change in the material necessitated a change in the form. The rack took the shape of a ladder, the sides being formed by angle-irons and the rounds being the teeth or cogs, which were made of round iron. This railway was completely finished in 1869, and since then, up to the present hour, for a period of over 15 years, it has safely conveyed thousands of tourists to the lofty summit of Mount Washington without the loss of a life.

The first sections of the wrought-iron rack had hardly been laid at Mount Washington, when tidings of what Marsh was doing in America reached Switzerland, that world-famed mountain region. Shut in by, and at the same time separating, the leading commercial countries of the European continent, Switzerland had always been greatly benefited by the international traffic across its snow-capped mountain ranges. Keenly awake to this fact, she had always jealously watched her interests, and with the demands of growing trade, the bridle-paths and the mule caravans of a by-gone age gave way to magnificent road structures with the modern freight wagon and the stately post coach. The roads over the Splügen, the Bernhardin, the Lukmanier, the St. Gothard and the Simplon, are masterpieces in the art of road-making, and the boldness of some of their parts will always excite the wonder and the admiration of the traveler. But just then the public mind was in a state of growing excitement. The Brenner Pass in Austria, near to the eastern frontier of Switzerland, had just been opened to public traffic, and the whistle of the locomotive echoed and re-echoed merrily along the rocky line. Away down in the southwest, drilling machines were diligently at work in the heart of the Mont Cenis, making the big hole, where Fell had the audacity to climb the mountain even during the construction of

* See descriptions of Mount Washington Railway, by Charles W. Copeland, M. Am. Soc. C. E., and Francis Collingwood, M. Am. Soc. C. E., in the Proceedings of the American Society of Civil Engineers, Vol. III, page 12 (February, 1877).

the tunnel. These were threatening clouds for the vested rights of the little republic, and something had to be done to avert the danger. Projects of all sorts for working steep gradients were freely discussed, and it was a well-timed thing that the Swiss representative in Washington should send to his government an official report of what Marsh was quietly doing in the White Mountains. This happened somewhere in 1867, or the early part of 1868. It was clear that a railroad had to cross the Swiss Alps, and the St. Gothard Pass, on account of its central location, and notwithstanding its serious engineering problems, was the one most prominently discussed. In view of the prospective large traffic, and also of the shortcomings of regular adhesion service, it was not thought judicious to carry the gradients beyond 1 in 40, or thereabouts. But this was equivalent to an expensive development of the line and to a very long tunnel. It was in the autumn of 1868 that a proposition was made to adopt a ruling gradient of 1 in 20, and use the rack and pinion. This proposition came from N. Riggenbach, master mechanic of the Swiss Central Railway, conjointly with O. Zschokke, civil engineer, Though the adoption of the rack might have resulted in a saving of about one-third of the total cost of the road, yet the entire absence of practical experience on a line of this importance decided its rejection, and the St. Gothard was reserved for the adhesion locomotive.

The stone was rolling. The air was filled with enterprise, and if it would pay to run a railway up Mount Washington there would be tenfold less risk in doing the same thing on the Rigi, that most renowned and most frequented elevation of the world. In the spring of 1869 an engineer left for America to prepare a minute professional report on Marsh's enterprise. The necessary capital for the Rigi Railway was soon afterwards secured and operations were commenced. Thus Riggenbach introduced the wrought-iron "ladder-rail" in Europe in his Vitznau-Rigi Railroad, which was opened in 1871. All the details of this line are nothing but modified copies of the Mount Washington road. Engine and rack-rail—the principal dimensions being retained—were built stronger, more substantial, and with more care throughout. The round teeth, however, of the original ladder-rack were here abandoned in favor of a trapezoidal tooth and the system of involute gearing. The maximum gradients on the Vitznau-Rigi road are 1 in 4, and, like the Mount Washington Railway, it is essentially a pleasure line, serving to convey tourists up and down the mountain sides.

About the time that work was commenced on the Rigi Railway, Riggenbach started also to build a short line connecting the sandstone quarries of Ostermundingen (Berne) with an already existing adhesion line. The same wrought-iron ladder-rack was employed as on the Rigi enterprise, but in other respects there were essential differences. On the Rigi the rack-rail is laid throughout the entire length of the line; here, however, we find the line made up of rack-rail and of pure adhesion sections, the rack-rail being only used where the gradients required large traction. The maximum gradient is 1 in 10. The locomotives were of such a construction that either the pinion or the adhesion drivers could be thrown in or out of connection with the steam cylinders, and the train could thus be either propelled by adhesion alone, or by the pinion and rack alone, according to the gradients of the road. This line has remained in successful operation to the present day, and, excepting the short-lived attempt on the Madison and Indianapolis Railroad, we have here the first instance of a practical combination of the rack-rail with pure adhesion portions.

The immense financial success of the Rigi Railway, together with the fact that thousands upon thousands from all parts of the civilized world assisted every year to spread the fame of that remarkable spectacle of the puffing locomotive on the mountain side, soon led to the construction of other rack-railroads. The Mount Washington road had given the first impulse, the Rigi did the advertising, and the line to the quarries of Ostermundingen pointed the way to every-day applications. The total aggregate of the lengths of rack-railways laid in Europe alone up to the present time is about 28 miles, but it must be remembered that the Mount Washington line, which only has a length of 2.8 miles, if built with a uniform gradient of 1 in 40 would be over 27 miles in length, and would have to be run at 29 miles an hour to be the equivalent in time of the Mount Washington Railway at 3 miles an hour. The following is an enumeration of the lines thus far built, all of them being still in regular operation.

RAILWAYS WITH RACK-RAIL LAID OVER THE ENTIRE LINE.

	CONSTRUCTED.	MAX. GRADIENT.			RADIUS OF CURVES.
					Feet.
Mt. Washington, New Hampshire	1866-69	1 in 2.67	37.5	per cent.	554
Vitznau-Rigi, Switzerland	1870	1 in 4	25	"	590
Kahlenberg-Vienna, Austria	1872	1 in 10	10	"	590
Schwabenberg-Pesth, Hungary	1872	1 in 10	10	"	590
Arth-Rigi, Switzerland	1874	1 in 4.8	20.8	"	590
Rio de Janeiro, Brazil	1882	1 in 6.7	15	"	
Drachenfels to the Rhine, Germany	1883	1 in 4.5	22.2	"	

RAILWAYS WITH RACK-RAIL AND ADHESION SECTIONS COMBINED.

	CONSTRUCTED.	MAX. GRADIENT.			RADIUS OF CURVES.
					Feet.
Ostermundingen, Switzerland	1870	1 in 10	10	per cent.	790
Rorschach-Heiden, "	1874	1 in 11	9.1	"	395
Wasseralfingen, Germany	1876	1 in 12.5	8	"	345
Rüti, Switzerland	1877	1 in 10	10	"	
Laufen (Quarries), Berne, Switzerland	1878	1 in 16.7	6	"	345
Oberlahnstein, Germany	1880	1 in 10	10	"	

The railways with rack-rail and adhesion sections combined, as a rule are worked in the manner described with the Ostermundingen line, that is, the engine exerts traction either by adhesion or by the pinion alone; yet in several instances attempts have been made to utilize adhesion simultaneously in combination with the pinion by simply coupling the adhesion drivers with the pinion, and notwithstanding the grave practical objections which stand in the way of such an arrangement, locomotives of this type can still be found.

These lines gradually and unmistakably demonstrated not only the feasibility, but the great practical bearing of the rack-rail for working steep gradients. They showed that inclines could be safely operated, which with any failure in the mechanism would of necessity lead to terrible catastrophes; but so reliable were the provisions against such an event, that the first accident has yet to be recorded. They also showed that sharp curves could be rounded with ease, and, irrespective of the

state of the weather, or of the conditions of the track, the maximum train weight could always be hauled over the line.

In the meantime work had been progressing on the long tunnel through the St. Gothard. The cost was greater than had been anticipated; and in 1876, when work was to be commenced on the approaches to either side of the tunnel, the company found itself in very straightened circumstances. Once more the application of the rack-rail and the adoption of steeper gradients was discussed, and this time with great seriousness. It was expected to finish the road with the means at hand. A plan was worked out. The project was warmly supported by several prominent continental engineers; but, though the feasibility of the system was acceded to in principle on all sides, no definite conclusion could be arrived at, and the subject was ultimately tabled on account of the resistance of the Italian delegates, and a somewhat general reluctance to adopt the system in its then undeveloped state.

Defects of the Ladder Rack-Rail.

Following thus the historic developments of the means for working steep gradients, we have been able to see how, little by little, the rack-rail has established for itself a certain modern recognition—a recognition, too, which is founded upon stubborn facts furnished by experience. There can no longer be any doubt concerning the practicability of the rack as a mechanical device in all these applications; and wherever once introduced in its modern form it has kept its own ground, thus furnishing a proof of the economy of its use. Yet withal it must be recalled that the existing rack-railroads are of moderate extension, of moderate traffic, and are operated at moderate velocities only. When it really comes to the higher demands of a regular railroad service, we find that the ladder-rack of Sylvester Marsh has serious inherent defects which incapacitate it for such purposes, and for which defects the very form of the rack excludes all remedies. The prime requisite for a smooth and quiet motion in all gearing is uniformity of pitch; and the greater the velocities the more important is this uniformity. It is the recognition of this fact which has led to the elaborate gear-cutting and gear-milling tools of our days. But the very mode of manufacturing the ladder-rack, the punching of its sides, the riveting of teeth and sides, etc., introduces elements of error which, though small by direct measurement become, apparent at higher speeds. Then, also,

the ladder-rack is built in sections of about 10 feet, so as to reduce the influence of expansion by temperature. Still that influence is there, and it exists at every joint in form of an unequal pitch. The velocity on the existing rack-railroads, and on the steeper gradients, is generally from 3¾ to 4½ miles per hour. At 5 miles per hour, the operation of the pinion in the rack much resembles a sort of trip-hammer action; and every increase of speed tells in the rapid wear of the pinion, though made of the best of steel. With an inability to run at high speeds, the locomotive, though running slow on steep gradients, and working with its full boiler capacity, cannot run correspondingly faster on the rack-rail sections of easier gradients where the traction is less. The engine, therefore, is unable in such cases to exert its full power, and this involves loss of time and an increased cost of transportation.

In order that the motion of the pinion in the rack—even under the supposition of an absolutely uniform pitch—be free of blows or impacts, it is necessary that before one pair of teeth has gone out of contact, another pair must have come to fairly bear against each other. This important condition determines the size, *i. e.*, diameter, of the pinion for a given pitch. The size of the pitch again depends upon the kind of materials employed, the amount of traction, and the width of the teeth. In the present ladder-rack the pitch is already as small as it possibly could be chosen, and still the necessary diameter of the pinion becomes so large, that, together with the low speed, the power from the steam cylinders has to be transmitted to the pinion by multiple gearing. But multiple gearing complicates the locomotive considerably; it introduces additional frictional losses; its renewal from time to time is expensive; and the additional rotating masses between the pinion and the elastic cushioning steam are not conducive to a soft action.

In the present ladder-rack the strains are also very high—especially so on the curves; in fact they are greater than what generally would be considered a safe working strain. A radical increase of traction, as the heavy traffic of a first-class regular line might demand, is altogether beyond the capacity of the ladder type. Considering the inequalities of pitch due to manufacture, and still more the discontinuity of the rack, its 10-feet sections, and the chance of slight derangements, it becomes evident that an increase of traction by means of the direct coupling of two driving pinions is a thing utterly impossible. Any strengthening of the ladder-rack by an increase in the size of the pitch would only

be creating most serious difficulties of another kind, recollecting what has been said about multiple gearing.

The ladder-rack has done well enough in its present sphere and under its present conditions, though even here suffering from its constitutional troubles; but when the imperious demands of an ever-progressing age are to be satisfied, then the ladder principle sadly fails. Its defects are inherent; they are of the blood and flesh of its form; and to free the sound and healthy rack principle from those drawbacks, and expand its usefulness, required the discovery of an entirely new type. This, we venture to say, has been admirably accomplished by Mr. Roman Abt. Mr. Abt has been for many years chief constructing engineer in Riggenbach's various rack enterprises. Many of the applications of the rack-rail in Europe were made under his immediate care, and, if any one has been able to thoroughly study and master the subject, he has. His new rack system embodies the experience of years in this class of engineering, and it is the writer's opinion that the rack, which thus far had been regarded more as a sort of makeshift, to be applied in extraordinary cases which required extraordinary means, has been advanced by him into a most important element in the planning of new roads, or in the economical operation of old ones.

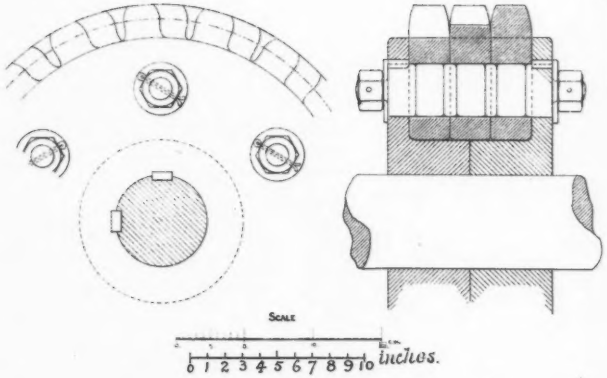
Abt's Rack-Rail. (See Plate V.)

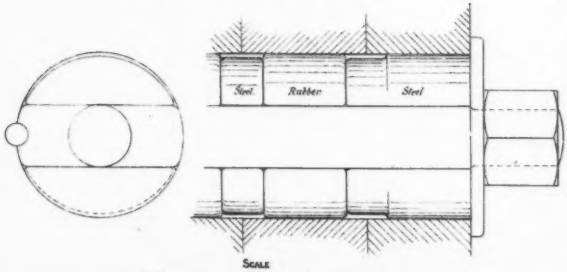
In the construction of his new rack-rail, Mr. Abt has entirely abandoned the hopeless ladder-type, with its two sides and its rounds or teeth in between. His rack-rail consists essentially and is built up of a number of individual elementary racks placed side by side. Each one of these elementary parts is nothing but a rack in the very simplest form, which is that of a bar of given length provided with teeth. Such an elementary rack will be able to stand a certain safe working strain, which is positively determined by the material, the dimensions of the bar and the pitch of the teeth. Whatever may be the traction to be developed by the pinion, by placing side by side a sufficient number of elementary bars, each one strained only to its chosen limit of working strain, we are able to meet any amount of traction as far as the strength of the rack-rail is concerned. These bars are connected among themselves by iron or steel chairs and bolts. The chairs, which are placed at regular intervals, also keep the bars laterally distanced from each other, and through these chairs the whole is firmly bolted or otherwise secured to the wooden or iron ties of the road-bed.

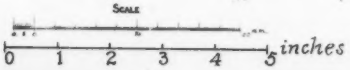
THE
ABT SYSTEM OF RAILWAY

(*Harz-Railway*)

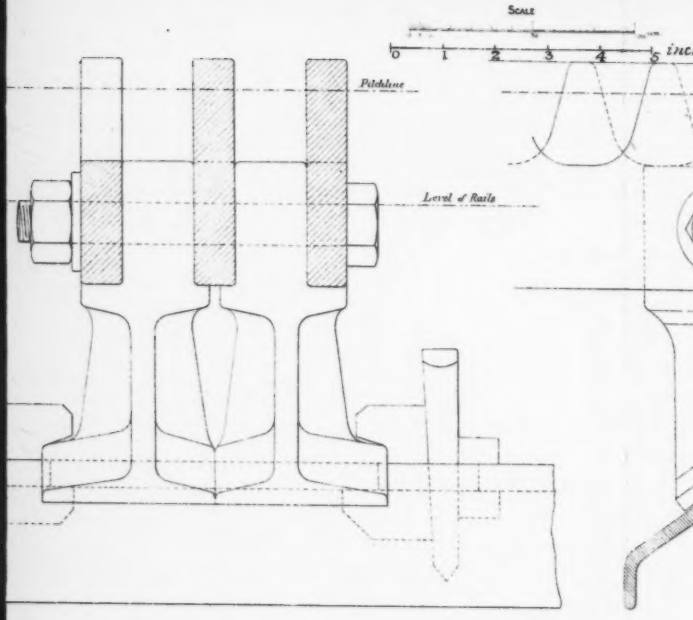
PINION



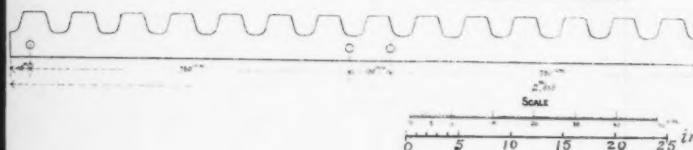




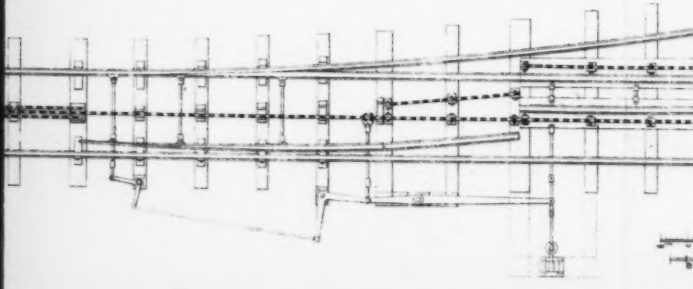
RACK RAIL & CHAI



RACK BAR

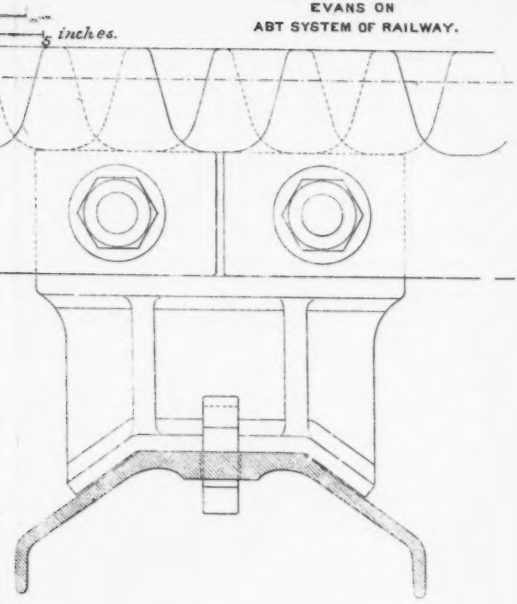


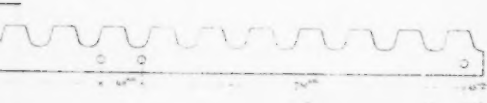
SWITCH

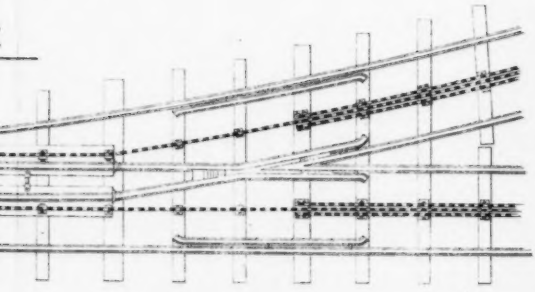


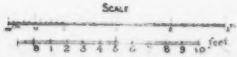
CHAIR

PLATE V.
TRANS. AM. SOC. CIV. ENG'RS.
VOL XV. Nº 323
EVANS ON
ABT SYSTEM OF RAILWAY.





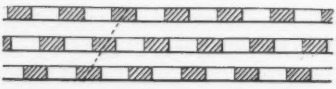




The material of the bars may be either rolled-iron, Bessemer steel, cast-steel, or eventually also a good quality of steel castings, and the choice will be governed by convenience, or by the requirements of the road. With rolled bars the teeth may be produced in manifold ways, be it by punching, drilling, milling, sawing, pressure in dies, etc.; but no matter how produced, the last finishing touch ought to be always given by milling, which secures a perfect uniformity of pitch, the one important point to be aimed at. It guarantees a perfect contact between rack and pinion, and with the facilities of the modern machine-shop its expense is no barrier to its application.

Mr. Abt lays the various bars, of which his rack-rail is composed, with broken joints. In this manner his rack-rail becomes practically one continuous structure, always one or more whole bars passing the joint of every two bars. Through this continuity of the rack-rail, and through the fastenings to the ties, a most desirable and important union is established between these two elements of the road, and still old ties may be exchanged for new ones without causing the slightest difficulty in the maintenance of the line. The breaking of the joints has the further advantage of equalizing throughout the influence of expansion by temperature. Comparing this with the ladder-type, with its discontinuous 10-feet sections, the advantages of Abt's rail with its continuous solidity and strength are very striking.

But the component bars of Abt's rack-rail are not only laid with broken joints, they are also, if the expression be permitted, laid with broken pitches—that is, the teeth of the side-by-side bars are arranged in the form of steps, as shown below, which is a beautiful feature of



Abt's rail. It enables the simultaneous contact of several teeth of the pinion with different bars of the rack-rail, thus giving a contact and a smoothness of motion that could never be obtained with the ladder-type. This interlocking of several teeth offers at the same time an increased security against the fracture of a tooth, and even if, from some external cause, such an event should occur, it could not be followed by any serious consequences. In discussing the defects of the ladder-rack, we have seen that for the very reason of getting just

sufficient contact, the pinion had to be made of a certain size, and with that size multiple gearing became unavoidable; through the stepping of the teeth, however, Abt's pinion can be made so much smaller in diameter, without interfering with a still superior contact, that multiple gearing is entirely unnecessary, and is dispensed with. The ensuing simplicity and efficiency of the locomotive is not the least of the many excellent points of Abt's system.

The pinion is constructed of as many separate disks as there are separate bars in the rack-rail, each disk corresponding to a line of bars, and the disks are shifted against each other according to the stepping of the teeth in the bars. Another very important feature in the construction of the pinion, is that its various disks are in an elastic connection with the shaft by means of inserted springs. (Plate V.) By this arrangement the different disks obtain a small, limited, relative motion to each other. We had occasion to point out the sources of error which would lead to imperfections in the manufacture of the ladder-rack; the manufacture of Abt's rail, on the contrary, admits at every step of the application of tools which secure accurate work. Yet whatever remains of slight inaccuracies, be it in manufacture, be it due to the mode of fastening on the ordinary ties, or be it due to expansion by temperature, they are most cleverly counteracted by an elastic connection of the pinion disks with the shaft; and an absolutely perfect, automatic contact of the teeth is established, with the pinion traction uniformly distributed over the various bars of the rack-rail. Practical experience has fully borne out the high expectations that were entertained in the conception of this system. Even at 16 miles per hour a motion is obtained on rack-rail sections which is perfectly smooth and free from any blows or irregularities. Not the slightest noise is to be observed, the pinion working as silently in the rack as the regular adhesion drivers on the ordinary rails.

A characteristic property of Abt's rack-rail consists in its economic accommodation to the gradients of a road. The numbers or the thickness of the individual bars of which the rack-rail is composed, are chosen according to the traction to be sustained; number or thickness of the bars, therefore, increase and decrease with the gradient. Such an accommodation is impossible with the ladder-type for obvious reasons.

Then, again, curves of any radius can be laid with ease, because the same bars are used as on the straight portions of the line. The ladder-

rack, on the contrary, has to be made on purpose, with great care, and for each radius separately.

On the Mount Washington, Rigi and several others of the earlier railroads, the rack-rail rests on wooden stringers, which in turn are fastened to the ties. With the introduction of the mixed, or combined adhesion and rack-rail, systems it became necessary to give to the rack-rail a sufficiently elevated position relative to the ordinary rails, so that the pinion could clear the latter safely at switches and crossings. Iron chairs were now introduced for support. The free position of the rack-rail obtained in this manner is an excellent safeguard against accumulations of snow and mud. With Mr. Abt's system this advantage exists in a higher measure, on account of the narrow bars of the rack-rail and the comparatively wide clearances between them. These clearances form a kind of furrow, which admit of a mechanical cleaning, or, we might say, plowing, by a device on the locomotive itself. As mountain regions will particularly be the haunts of rack and pinion, and mountain regions generally have heavier snow-falls, it will be of interest to cite the experience of the existing lines. Ostermundingen and Rorschach-Heiden, in Switzerland, and Wasseralfingen-Friedrichsberg, in Germany, are all three operated summer and winter, and at all these places severe winters are prevalent. The lines were constructed respectively in 1870, 1874 and 1876; they were opened to traffic in the years immediately succeeding, and have been regularly operated ever since. Yet in all this time trains have run on these lines in the heaviest snow-storms, and not a single train has ever been stopped on account of snow or snow-drifts. The rack-rail locomotive is provided with a sort of snow-plow, and the positive action of the pinion always pulls it through. The Fourth Annual Report of the Rorschach-Heiden road says: "The extremely heavy snow-falls of last year not only did not influence the regular operations of the line, but we had no expense whatever for removing the snow. Experience shows that a heavy snow is far less obstructing to a rack railroad than to an adhesion line; no matter how large the accummulated masses are, with a rack railroad they are of no account." Practice has demonstrated that the rack locomotive can pass snow-drifts which the adhesion locomotive could not overcome. The rack-rail on the above lines belongs of course to the ladder-type, but no difficulties with snow having appeared there after so many years of constant service, none will exist with the superior Abt rack-rail.

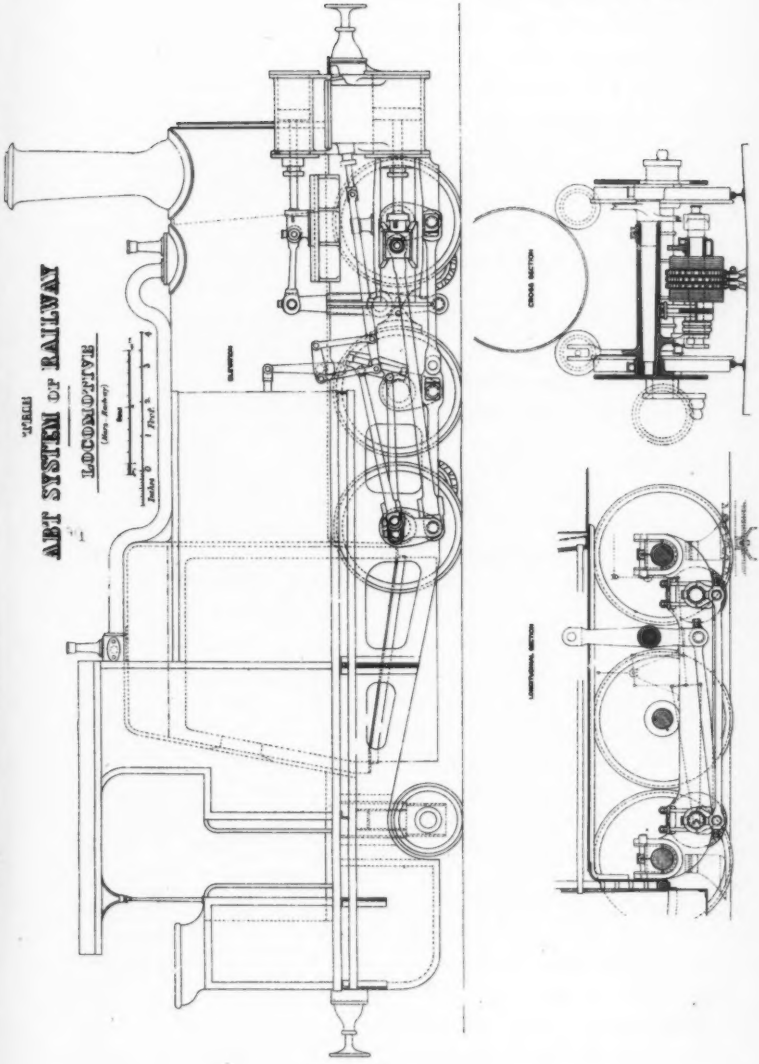
Crossings and switches are made and operated with rack-railroads similar to those on pure adhesion lines, and in every respect they are as simple and as safe. The first switches were constructed in the form of sliding-tables and turn-tables, as, for instance, on the Mount Washington, and immediately afterwards on the Rigi Railway, but they soon were built in close adherence to the customary adhesion devices. (See Plate V.)

The peculiar method by which, on a line of the combined system, the change is made from the adhesion portions to the rack-rail without stopping the train, will be described after having discussed Abt's locomotives.

Abt's Locomotives.

The rack-rail enables us to adopt steeper gradients in the development of our railroads; we can follow more closely the natural slopes and shorten our line. But by combining with the steeper gradients the easier ones of the pure adhesion system, we expand the usefulness of the rack-rail enormously, provided we are also capable of building locomotives which, under all the varying conditions of grade, can invariably exert their full power. This means that traction and corresponding speed must vary in inverse proportion: large traction with low speed, small traction with high speed. If we simply take a line with a maximum gradient of 6 per cent., and with other gradients ranging all the way down to level, the condition of a full exhaustion of the engine's power in every part of the line demands a most extraordinary capacity of accommodation. On adhesion portions an adhesion engine will do the work, and will do it well if properly dimensioned, but once those gradients are transgressed which practical experience has designated as a limit to an economical adhesion service, the locomotive necessarily must fail, not only on account of lack of traction, but also, and which is equally important, on account of lack of purchase for the steam. Weight, that recourse of regular locomotive practice, together with increased cylinder volume, does not offer any satisfactory solution, for while it slightly improves the climbing qualities, it introduces drawbacks in its stead by which the benefit derived has to be dearly paid for all over the line. With the rack and pinion, on the other hand, weight is practically immaterial for their working. We could therefore take any type of adhesion locomotive which, with comparatively moderate weight, has been approved of for practical adhesion gradients, and by adding the pinion

PLATE VI.
TRANS. AM. SOC. CIV. ENG'RS.
VOL XV. Nº 323
EVANS ON
ABT SYSTEM OF RAILWAY.



mechanism as an independent and separate part throughout—with an independent set of steam cylinders—we could add exactly those qualities which are indispensable in the mounting of the steeper gradients. This is precisely what Mr. Abt has done in designing his locomotives. Adhesion drivers work separately and independently from the pinion device; they work alone on the pure adhesion portions of the line, and they quietly keep on working when rack-rail sections are encountered. Rack and pinion then supply the additional traction, and the extra steam cylinders secure the necessary purchase for the steam. The speed adjusts itself according to the total traction and the steaming qualities of the boiler, and the engine develops its full powers under the most varying and trying of circumstances.

To the exhausts of the ordinary adhesion cylinders are added, when ascending rack-rail portions of the line, the more numerous exhausts of the pinion cylinders. This produces an energetic and uniform blast, and secures a free and abundant steaming at the very time it is most needed.

Another important reason for utilizing the natural adhesion of the locomotive, exists in the fact that, even on gradients of 600 and more feet to the mile, and with trains of a still practical size, the traction due to adhesion alone will be from $\frac{1}{2}$ to $\frac{1}{3}$ of the total traction. The rack-rail can thus be relieved of a corresponding amount of strain, and can be made lighter and cheaper. A certain engine weight we shall always have, for weight and the capacity of performing work are, to a certain extent, parallel terms in locomotive construction, and this necessary weight can be excellently made use of in relieving the strain on the rack.

The characteristic feature, then, of Abt's locomotives, consists in the additional mechanism with which any well approved or favored type of adhesion engine is equipped, and thus is transformed into a locomotive for a rack-railroad. Through Mr. Abt's ingenious rack-rail construction, which admits a pinion of small diameter, multiple gearing, as we stated previously, has become superfluous; the pinion shaft is driven directly from the separate pair of cylinders, and the construction of the rack-rail locomotive is reduced to a simple form.

The vast superiority of the independent adhesion drivers and pinion over their combination by direct coupling, as has been done on some of the modern rack lines, is evident. A rigid coupling of the two mechanisms presupposes an ever-constant relationship between the pitch-line

diameter of the pinion and the diameter of the adhesion drivers, a condition of things which practically it is impossible to maintain. Important frictional losses are unavoidable with such a design; then, besides, there being only one pair of cylinders—unless there should be a special second train of wheel-work—the engine lacks that capacity of economical accommodation which forms such an important feature of Abt's locomotive. In this engine of his, the separation of the adhesion and the pinion device is so absolutely perfect that neither one can disturb the other. Whether on account of the condition of the track the adhesion drivers slide or not, whether they be running ahead or behind, all this has not the slightest influence on the pinion, which steadily does its part.

It is true that on the engines of the Madison and Indianapolis Railroad the pinion was driven by independent steam cylinders, but Abt's engine is otherwise entirely different in construction. Then, again, Abt's whole railway system is essentially grounded upon his rack-rail, and his engine is the logical sequence of the latter.

Through Abt's rack-rail it has become possible to build rack-rail locomotives of special large traction, *by simply coupling two or more driving pinions.* All the objections which existed with the ladder-rack against the coupling of pinions as a means to increase tractive power, are completely overcome in Abt's rack-rail type. With its continuity, its perfect and equalized pitch, and the elasticity in the pinion disks, coupling has not only become practicable, but by placing one pinion a little in advance of the other in reference to time of contact, coupling only heightens the uniformity and smoothness of motion.

In his original engine design, Mr. Abt selected the Engerth type of adhesion locomotive. There were six coupled adhesion drivers. The tender carried the pinion shaft proper with the one driving pinion in the center. At either end of the pinion shaft were brake disks, to which the crank pins were attached. The tender frame supported the extra steam cylinders, and steam was conducted to and fro through a specially designed ball and socket coupling of locomotive and tender.

In his Harz locomotives (Plate VI; see also page 177), the first ones actually built upon his system, Mr. Abt adopted the adhesion type of a six-coupled tank engine, with a trailing axle behind the fire-box. They have two coupled driving pinions placed under the front part of the locomotive, and the additional pair of cylinders is arranged above

those for the adhesion drivers. The two pinion shafts rest in a frame hung to the first and third of the adhesion axles. Motion is transmitted from the cylinders above to the pinions below by means of rockers, in a very compact and clever manner. The size of the cylinders is small, as the piston velocity, by reason of the small pinions and the interjacent connections, is large. The frame of the pinion shafts, being suspended directly to the adhesion axles without any intervening springs, establishes the proper relative position of pinion and rack. The constant relative position between the rack-rail and the ordinary rails is preserved through the regular maintenance of the line, which experience has demonstrated is easily done. Another protection against any incorrect contact between the teeth, on account of a variation in the depth of interlocking between pinion and rack, exists in the system of involute gearing itself. This system possesses the valuable quality of the contact remaining always correct, though the depth of interlocking may undergo variations. The natural wear of the adhesion tyres is compensated from time to time by a readjustment of the bearings.

The link gear is, of course, separate for each of the two sets of steam cylinders. For the adhesion cylinders the starting levers or screws are arranged in the usual manner, with a horizontal motion, whilst these levers or screws for the incidentally working pinion cylinders are given a vertical motion; thus well discriminating between the two.

The braking arrangements are very efficient. There is first an ordinary brake for the adhesion drivers; then there are brake disks on the crank shaft of the pinion mechanism, by means of which the pinions may be completely locked; and thirdly, there is an auxiliary loose pinion, with brake disks, on the trailing axle of the locomotive. The last one serves merely for the purpose of stopping the train in case of any accident to the driving pinions or their gear. The braking on down gradients is done by using the steam cylinders as air-compressors, there being special provisions for this purpose. This is the most perfect brake imaginable, and it is used on every one of the existing rack-railroads. In regular service all down-hill braking is entirely done by the driving pinions of the locomotive, and whatever braking pinions in the train are being called upon—one or more cars being provided with such. Wherever, therefore, the rack-rail is laid, the wear and tear of braking is taken off from the rails and tyres without proportionately diminishing the lives of rack-rail and pinion, because the sliding and rolling of the

teeth in contact occurs on smooth and well-greased surfaces. The work of braking is chiefly spent here in the compression of air in the steam cylinders, while with the usual braking device, *i. e.*, brake shoes pressing against the wheels, it is busily engaged in the demolition of rails, tyres and shoes.

Mr. Abt, when designing the normal type of his engines, which were to run on lines connecting directly with existing adhesion railways and using the regular rolling stock, had to limit the net tractive power to the working capacities of the draw-bars and the buffers in use. The normal draw-bar of the European lines possesses a working pull of 14 300 to 15 400 pounds, while the normal buffers admit a working compression from 16 500 to 17 600 pounds. Mr. Abt, in order to get as powerful an engine as possible, chose the latter figure of 17 600 pounds, and the locomotive when exerting the maximum net traction has its place at the rear of the train, and pushes it ahead. Besides an increased safety in the operation of lines with heavy gradients, there are still other reasons, which we will touch further on, why the engine is preferably placed to the rear.

The 17 600 pounds then represent the net traction of the locomotive; whatever traction is necessary to move the engine itself has to be added in order to obtain the gross traction; that is, the combined effort of pinion and adhesion drivers. It is evident that for lines with special rolling stock, or with cars having extra strong draw-bars, locomotives of far greater tractive power can be built.

The conditions for a powerful locomotive of this class are more favorable in our country, because the draw-bars are substantially stronger. Our draw-bars are capable of standing a safe working pull or push of at least 21 200 pounds, and we may adopt this figure for the present as the net traction of the normal American types of Abt's engines. These engines will weigh 50 gross tons. The following traction table gives the net train weights in gross tons, exclusive of locomotive, which these engines will haul on the various gradients; it also gives the corresponding speeds in miles per hour.

TRACTION TABLE A.

GRADIENTS.			TRAIN WEIGHT.	SPEED.
Per Cent.	Ratio per Unit of Rise.	Feet per Mile.	Gross Tons.	Miles per Hour.
3.	1 in 33⅓	158.4	290	8.6
3.5	1 " 28.6	184.8	250	8.4
4.	1 " 25.	211.2	222	8.2
5.	1 " 20.	264.	180	7.8
6.	1 " 16⅔	316.8	152	7.5
7.	1 " 14.3	369.6	130	7.2
8.	1 " 12.5	422.4	115	6.9
9.	1 " 11.1	475.2	102	6.6
10	1 " 10.	528.	92	6.4
11.	1 " 9.1	580.8	84	6.2
12.	1 " 8¼	633.6	77	6.
13.	1 " 7.7	686.4	71	5.8
14.	1 " 7.1	739.2	66	5.7
15.	1 " 6⅔	792.	62	5.5

On pure adhesion sections the speed of these engines may be run up to 30 and 35 miles per hour, while on rack-rail sections of lighter gradients a velocity of 12 miles per hour can safely be maintained in regular service. And such a speed we may need even on rack-rail sections if the system is to answer all the possible requirements of a regular traffic in passengers and merchandise.

When comparing our traction table with existing tests of adhesion engines, it is to be remembered that the table is based upon a net pull or push at the draw-bar of 21 200 pounds. Any larger admissible draw-bar strain will enable us to build our engines with proportionately greater traction, so that they will haul heavier trains. Another point worthy of being considered is, that our table does not present holiday dresses nor parade performances with exceptional conditions of track and sand-box assistance, but the figures can be relied upon as the regular every-day train-weights, irrespective of the weather or track. When making this statement we have not lost sight of the functions of the adhesion drivers, and we may add in explanation that their apportioned traction is based upon a low coefficient of adhesion. When the engine performs its maximum duty, then the steam is working at half-stroke admission.

The enormous adhesion engines which have been built of late, make

very severe demands on the capacities of the draw bars and the strength of the cars, in some instances the draw-bar strain rising to 27 500 pounds. We thought it might be instructive to add a second Traction Table, B, based upon a net traction of 27 500 pounds, the engines weighing again 50 gross tons.

TRACTION TABLE B.

Per cent.	GRADIENTS. Ratio per Unit of Rise.	Feet per Mile.	TRAIN WEIGHT. Gross Tons.	SPEED. Miles per Hour.
3.	1 in $33\frac{1}{3}$	158.4	410	6.4
3.5	1 " 28.6	184.8	355	6.3
4.	1 " 25.	211.2	314	6.2
5.	1 " 20.	264.	254	6.
6.	1 " $16\frac{2}{3}$	316.8	213	5.8
7.	1 " 14.3	369.6	184	5.7
8.	1 " 12.5	422.4	162	5.5
9.	1 " 11.1	475.2	144	5.4
10.	1 " 10.	528.	130	5.2
11.	1 " 9.1	580.8	119	5.
12.	1 " $8.\frac{1}{3}$	633.6	109	4.9
13.	1 " 7.7	686.4	101	4.8
14	1 " 7.1	739.2	94	4.7
15.	1 " $6\frac{2}{3}$	792.	87	4.6

To build the engines for such a traction is easy enough, if only draw-bars and cars will stand the pull or push of 27 500 pounds.

It is self-evident that engines of smaller weight and power can be constructed to suit any requirements of traffic. For roads of light traffic and smaller trains, a special engine has been designed by Mr. Abt, with only one pair of steam cylinders for both adhesion drivers and pinion. In this case, of course, only one device can work at the same time. This locomotive resembles, in its action, the engines on some of the existing rack-railroads of the combined system, but the whole construction is by far simpler and better.

A very important question relates to engine repairs. Experience has taught that, even with the complicated engines with multiple gearing, the repairs are perfectly normal, and can be compared favorably to those of regular adhesion locomotives when doing the same amount of work.

AUTOMATIC ENTERING RAIL. (See Plate VII.)

On a line of the combined system it is of the greatest moment that the rack-rail be properly entered by the pinion without slackening the speed of the train, especially if the line happens to consist of a great many alternating sections. Mr. Abt accomplishes this by laying down a special entering rack-rail. This entering rail, of about 10 feet in length, is hinged upon the end which joins the continuation of the rack, and its other free end rests upon strong spiral springs. The teeth toward the free end gradually diminish in height, giving that end a wedge-shaped appearance when looking at it from the side. When the locomotive passes over the entering rail, steam is not yet admitted to the extra pair of cylinders, so that the pinion upon touching the yielding entering rail, and on account of the friction produced thereby, can turn about its axis, and within a few feet of distance traveled over, a true interlocking of the teeth is reached. Should the friction between the pinion and the entering rail be too small to produce a rotation of the pinion, then the latter will slide along the entering rail, and after a few inches of sliding the proper contact is secured. The very simplicity of this apparatus is the best guarantee of the absolute certainty of its automatic operation. It never has failed, and it works without any blows or noise.

When reflecting upon the working of the locomotive all along the line, apprehensions might be felt at first glance on account of the apparent complexity of the parts, and there being but one driver to take care of and manipulate the whole. But it must be considered first, that the locomotive is almost invariably placed in the rear of the train, and, secondly, that the various manipulations come about at different times and never simultaneously. With regard to the first point, one of the most trying and exhausting of the duties of the driver on a regular adhesion locomotive consists in the watching of the line with all its flying pictures, crossings, curves, signals, etc., and particularly so when the train is a fast express. His attention is divided between engine and line, and the greater part is absorbed by the latter. With the engine at the rear of the train, the line is watched by the brakeman on the front car, and the driver, thus greatly relieved, has mainly to care for his engine. On the adhesion portions of the line he has nothing but the adhesion parts to look after. When approaching a rack-rail section the speed of the train is moderate, the entering rail being passed at a speed of 5 to 6 miles

per hour, and he has ample time to watch the correct entering of the pinion before he gives steam to the extra set of cylinders. His adhesion engine needs no attention whatever at that moment, but is working steadily along. On down-hill gradients again all he has to care for are his brakes. There is only one thing to do at a time and plenty of time to do it well.

Having described the essential features of Abt's rack-rail and engines, it remains yet to say something of the practical applications of his system.

Applications of the System.

The location of railroads in mountainous districts has ever been one of the most difficult tasks of the engineer. Comprehensive knowledge, ingenuity and skill have planned many a wonderful line over mountain ranges and ridges, but the victories invariably had to be dearly paid for, and more than one of them has been a defeat for the unfortunate shareholder. Hemmed in by narrow precipitous valleys, winding their sinuous course around projecting mountain sides, sometimes abruptly changing their average gradients in steep ascents, the question of cost of construction is intimately associated with the limit of gradient which the prospective traffic may warrant for an economical management. Here each special case will present peculiarities of its own, but for important trunk lines or main arteries, like those that cross the Alpine chains of Europe, experience, running clear down to the present days, proclaims a gradient of 1 in 40, or $2\frac{1}{2}$ per cent., as the one yet compatible with an economic and safe management of a pure adhesion road. The Semmering and the Brenner are both built with this ruling gradient. At the Mont Cenis, to be sure, we find a 3 per cent. gradient for a considerable portion of the line, yet on the St. Gothard Railway, soon afterward, it was not thought judicious to go beyond $2\frac{6}{10}$ per cent. or 1 in $38\frac{1}{2}$, and most of the line follows 1 in 40. Higher gradients not only make severe demands on the tractive power of the locomotive, but, especially in mountain regions, where local dampness and frequent moist deposits materially reduce adhesion, they threaten also a safe and regular operation of the line, and this is one of the gravest objections against the application of steeper gradients for adhesion service. The lower the limit of gradient, the more serious will be the conquering of a rapidly ascending valley, or a sudden rise to some table land above. Instead of being able to follow the cheaper course along the water lines or valley bottoms, we

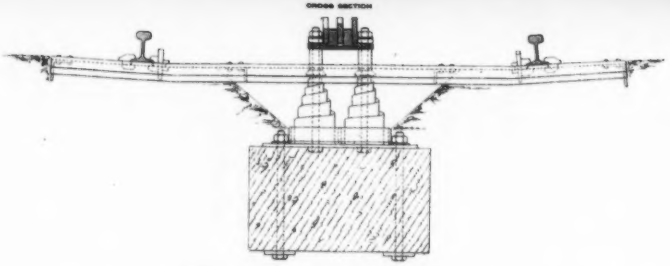
THE ABT SYSTEM OF RAILWAY

ENTERING RAIL

(*Hars–Ambrey*)

Scale

0 5 10 15 20 25 inches



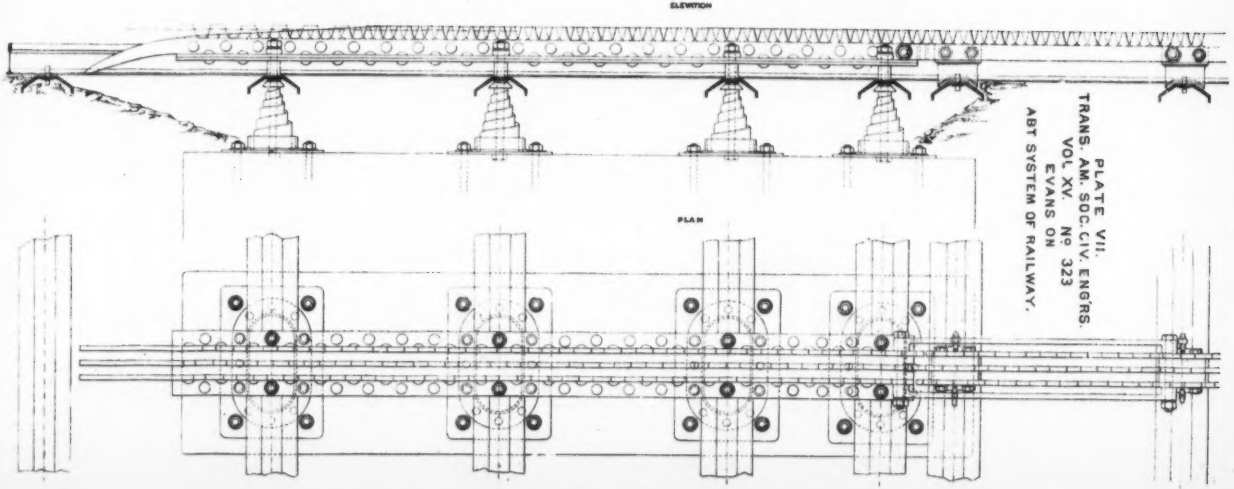


PLATE VII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XV. No. 323
EVANS ON
ABT SYSTEM OF RAILWAY.

often have to go way back from the sudden ascent, and begin at a point from which, by rising along the mountain sides with expensive cuts, retaining walls and costly tunneling through projecting spurs, we finally reach our aim with 1 in 40, or whatever be our limit. Sometimes a convenient side valley will give a chance for lengthening out the line along its sides, or by switching to and fro in zig-zag line along the slopes, we may obtain the length the gradient requires. In other instances we avoid the delays of the switch-back system by crossing and recrossing a valley from side to side, and in continuous long-stretched loops steadily make the ascent. Then again we boldly cut into the mountain itself, and develop the line in spiral tunnels, the paths often crossing themselves in their plans. A most remarkable example of this latter kind is found, for instance, near Giornico, on the Italian slope of the St. Gothard line. There the road winds itself in two spiral courses up, through, and in the precipitous mountain side, and though the point of ingress and the point where the line last leaves the tunnels are several hundred feet above each other, yet their horizontal distance does not much exceed 40 feet. Still, notwithstanding all such exertions, we often in crossing a range cannot obtain a sufficient altitude, and have to resort to a tunnel of unusual length and cost.

The lower the gradient to which the engineer is limited, the longer will be the line, the greater the call for artificial structures, the deeper the cuts, the higher and costlier the embankments, the more numerous the bridges, and the longer and more frequent the tunnels. Any system which admits an expansion of the limit of gradients will greatly reduce the cost of construction of new roads, but a system like that of Mr. Abt, which combines at once an extensive range of gradients with perfect safety and economy of operation, will prove a perfect boon, one of the most valuable gifts to our modern means of transportation. Through Abt's railway system we can adhere more closely to the bottom of the valleys; in fact we can take the ground much in the shape it is in. All our construction expenses in excavation of earth and rock and in masonry per mile of road will be smaller, the road itself will be materially shorter, and it will take much less time to build it and have it in operation. So great are the savings from all these sources, that, even including the extra expense for rack-rail and engines, the cost per mile of the shorter Abt railway will never reach the cost per mile of the longer adhesion line. Abt's system opens new and vast fields to railway

enterprise; and roads which never could have been thought of on account of enormous expense, can now be built and operated at so much lower cost that they will prove safe and paying investments to capital.

The selection of gradients is a matter of study in each special case. It is as much a function of the natural inclines of the ground as it is one of the general importance of the line, its traffic, and also its total length. On a regular adhesion road with a gradient of 1 in 40, an engine of 48 gross tons in weight will haul on an average a train of 180 gross tons, and the velocity will be from 10 to 14 miles per hour. Our traction table shows that an Abt locomotive can manage the same train of 180 gross tons on a gradient of 5 per cent., or 1 in 20, so that we can say at once, a line with 5 per cent. gradients and the rack-rail handles traffic as readily as one of 2½ per cent. gradients without the rack-rail. No extra engines are required, the same locomotives pull the same train over the entire road.

A principal line, therefore, with 5 per cent. gradients, built and operated by Abt's system, is in every respect equivalent to a regular adhesion line of 2½ per cent. gradients, as far as the handling of the traffic is concerned. But in order to rise from one level into another a line of 1 in 20 will only be half as long as one of 1 in 40.

According to the lengths of the lighter and heavier gradients on a given line, it can be demonstrated that even for principal railways a maximum gradient from 6 to 7 per cent., 317 and 370 feet to the mile may be adopted, and, as a rule, the same traffic be economically managed which is now handled by adhesion lines of 2½ per cent., or 132 feet to the mile. The number of cars, their construction, and the number of locomotives would be the same in either case. The train weight on a 7 per cent. gradient to be sure would only be 130 gross tons, but the proportionately higher speed on the lesser gradients, the resulting saving in time, and the ability of increasing the daily trains in number, fully compensate for the smaller trains.

The velocities on the rack-rail portions, though they appear small at first sight, are, in reality, similar to the velocities on pure adhesion lines. On a 7 per cent. gradient, for instance, our normal rack-rail locomotive will haul a 130-ton train with a speed of about 7.2 miles per hour, but this is equivalent to a speed of over 20 miles per hour on a gradient of 2½ per cent.

A very important point in favor of this new system when compared

with the mountain lines of usual gradients, is the materially increased safety in the working of the line. The hold of the pinion is absolute and positive, the adhesion is only utilized to what the average conditions of the track will warrant, and the facilities for braking are abundant and superior. Then again, the facility with which the trains can pass from adhesion sections to rack-rail sections, admits of arranging stations, depot-yards and switches on horizontal ground, and upon the pure adhesion system. Where desirable, rack-rail switches may be used on gradients, and can be operated with perfect safety.

The general operating expenses are lower with Abt's railway system than with the equivalent adhesion system. The rack-rail sections are considerably shorter than would be the adhesion sections which they replace, and there is correspondingly less maintenance and supervision of the line. There is, then, the additional rack-rail. The life of the rack-rail can be assumed from present experience to be 40 years with a very severe traffic passing over it. Calculating its renewal upon such a basis, and adding the cost of maintenance and ordinary care, which latter expenses are also drawn from the long experience with the unfavorable older form of the ladder-rack, we still get a considerable balance in favor of the rack-rail portions. The consumption of fuel, the expense of firing up, and the losses through stoppages at the stations are the same with either system. But the consumption on the open line is in favor of Abt. The work of lifting train and engine from one level to another one, is, of course, identical in either case; with Abt, however, the higher level is reached by a much shorter route, and the frictional losses, which are in direct proportion to the length of the line, are therefore smaller, and require less fuel to be overcome. It may be claimed that the application of a pinion and rack introduces frictional losses which are wholly unknown to the adhesion system. This claim holds good as far as the friction of the teeth is concerned, but when considering and calculating the frictional losses of the adhesion drivers in curves—and curves preponderate in mountain districts and hilly sections—and when further considering the losses which are due to the unequal wear of the tyres, which happen all along the line, it will be found that the pinion is still a more efficient worker than the ordinary adhesion driver.

With principal or main lines we have thus far spoken only of the application of 7 per cent. gradients; yet circumstances may even here

carry us far higher. If the chief ascent, for instance, occurs at a relatively short section of the line only, we may follow similar examples in the working of adhesion lines, and employ an auxiliary engine. With one engine pulling, and the other one pushing the train, we can carry the same load of 180 gross tons which is hauled on a 2½ per cent. gradient, up an incline of 1 in 10, or 528 feet to the mile. This is certainly doing remarkably well.

In fact two engines are capable of hauling a train of 124 gross tons, or six loaded cars of about 46 000 pounds each, on a gradient of 15 per cent., or 792 feet to the mile = 1 in 6⅔.

Existing railways with gradients of 150 feet per mile and upwards, will be highly benefited by laying a rack-rail on such gradients. This is an extremely simple matter which requires but very little time. The existing engines can either be rebuilt, or they can be replaced by new ones. The adoption of the rack-rail on such old lines resolves itself into an increased regularity and safety in the operation of the roads, an essentially increased capacity for hauling freight, and sensibly reduced working expenses. It may be stated in a general way that one locomotive of Abt can be made to do the service of two or three adhesion engines. In view of the heavy gradients on many lines of the American continent, it is well to call attention to these things.

With a smaller traffic we make use of smaller engines, and with these latter we can introduce a relatively larger traction, *i. e.*, be able to haul relatively heavier trains. Our traction table relates to what we termed normal types, weighing in complete working order 50 gross tons. In reference to small-sized locomotives, we may mention that an Abt engine is now in course of construction which is to work on a network of narrow-gauge tracks in slate quarries in Bavaria, Germany, and which in complete working order will only weigh 4½ gross tons. We shall speak of that line further on.

Local roads, and especially private lines to factories, quarries, mines, etc., are almost prohibited in hilly or mountainous districts by reason of their first great cost. Economy can only be maintained in a substantial manner by following closely the undulations of the ground, and this is precisely what Abt's system, with its enormous range of gradients, permits us to do.

A very advantageous and extensive application of the rack-rail will be found in connecting factories, mills, etc., with a neighboring

railway station, when either above or below its level. Railroad cars can then be run to and fro in the easiest and cheapest manner on a short rack-railway—a simple truck takes the place of a locomotive, and it is operated by either hand or steam power.

The Harz Railway.* (Plate VIII.)

The "Harz" is the collective term for a massive group of elevations belonging to that last chain of mountains beyond which extend the comparatively low lands and plains of northern Germany. Its medium elevation is about 2 000 feet, and politically it is situated in the Commonwealths of Brunswick and Prussia. Valleys, in the usual sense of the term, there are none to speak of; they partake more of the nature of romantic ravines or gorges with very precipitous slopes. The district contains many valuable mines and quarries, and it abounds in rich woodlands and abundant water power for industrial purposes. Of late years its isolation, by reason of poor means of transportation, made itself more and more felt, and its industries were seriously threatened by not being able to keep their own in the general competition of trade. Several railroad projects were worked out, one strongly supported for strategical reasons, a potent commendation in modern Germany; but they all had to be abandoned on account of the excessive cost of construction. The extent of the Harz district is not large, its greatest dimension being about 20 miles. A through railway, therefore, with its mountainous character and greater cost of operation, can never compete in through traffic with the cheaper, though longer, encircling lines, and it can only count upon the home traffic of the Harz itself. This latter is not large enough, nor will it ever be, to warrant the expense of an ordinary adhesion railway of 1 in 40, or even 1 in 35. Most careful statistical researches of the annual traffic were made, but things looked hopeless enough until Abt's system offered a satisfactory financial solution. The difficulties in locating an adhesion line were greatly aggravated by the fact that the line had to touch the greatest possible number of the scattered industrial settlements, irrespective of the most favorable route from an engineering point of view. All these difficulties disappeared with the freedom Abt's system admits as to gradients.

*In the description of this first actual application of the Abt system, we follow mainly a very able paper (see Glaser's Annalen für Gewerbe und Bauwesen, 1883) by A. Schneider, C. E., Herzoglich Braunschweigischer Bahndirector, who has made all the preliminary studies, and under whose immediate care the road is being built.

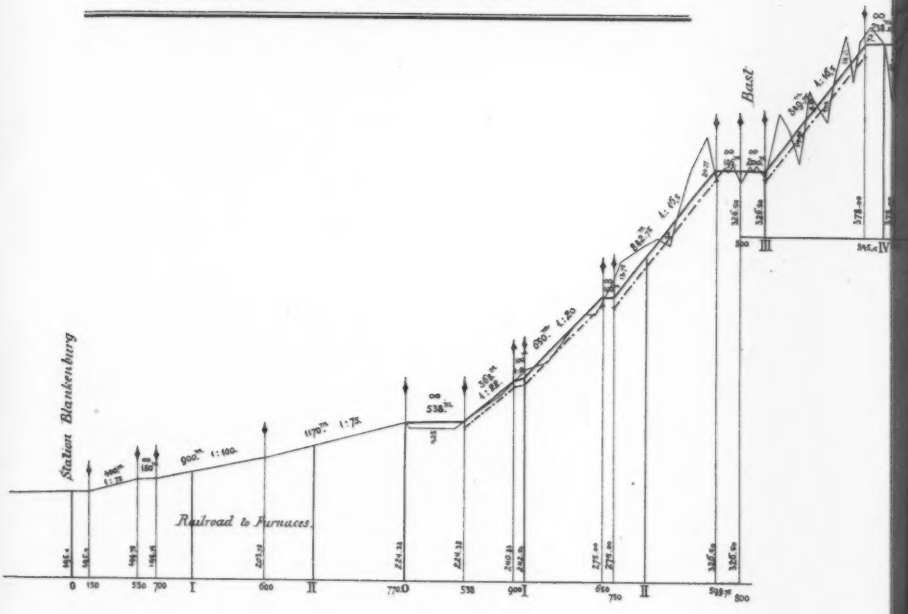
The line which is just now in the course of construction, and one-half of which will be opened to traffic this autumn, follows for about 1¼ miles an existing adhesion line from Blankenburg to the blast furnaces. Thence the new road rises rapidly for about 4½ miles to the mines of Hüttenrode. The difference in level is about 810 feet, and, with the exception of a few short horizontal sections, the gradients are 1 in 22, 1 in 20, and 1 in 16.5. From Hüttenrode the line descends with gradients of 1 in 40 and 1 in 100, with two sections of 1 in 20, a total descent of 315 feet in a distance of 2 miles; and ascending again, with gradients not exceeding 1 in 40, it passes the village of Rübeland, with its furnaces and smelting works; then Elbingerode, with its lime quarries, and, after a total rise of 415 feet in a distance of 4½ miles, the highest point is reached. In the farther descent to the blast furnaces of Rothehütte and the works near Tanne (the terminus) there are two sections with gradients of 1 in 18; otherwise the gradients are 1 in 40, 1 in 60, and 1 in 120, and less. The total length of the railway is not quite 17 miles, and 3.55 miles of this length are provided with the rack-rail. The maximum gradient for the adhesion sections is 1 in 40, and all steeper gradients are provided with the rack-rail. The maximum gradient of the latter is 1 in 16.5, or 320 feet to the mile. The line contains but very few straight sections, and forms in reality a succession of curves. This, in conjunction with the range of gradients, demonstrates forcibly the adaptability of this railway system to the exigencies of the ground. Along the open line the curves have a minimum radius of 655 feet, and in the village of Rübeland there is one of 590 feet. The rack-rail is laid with three elementary bars. Gauge of track, 4 feet 8½ inches. The total estimated cost of this line of 17 miles is $875 000, or about $51 500 per mile.

Extensive field studies and estimates were made in order to establish the difference in cost of construction between a line with pure adhesion gradients and one combined with the rack-rail. These estimates were made for the first 6.7 miles, 3.22 miles of which are laid in 8 different divisions with the rack-rail. These investigations include the minutest details, were carried through with the utmost care, and give the following results, showing the great economical value of the Abt system:

	Abt System.	Adhesion System.
Length	6.7 miles.	9.32 miles.
Real estate required	1 280 000 sq. ft.	1 780 000 sq. ft.

THE
ABT SYSTEM OF RAILWAY

HARZ-RAILWAY



PROFILE

SCALES

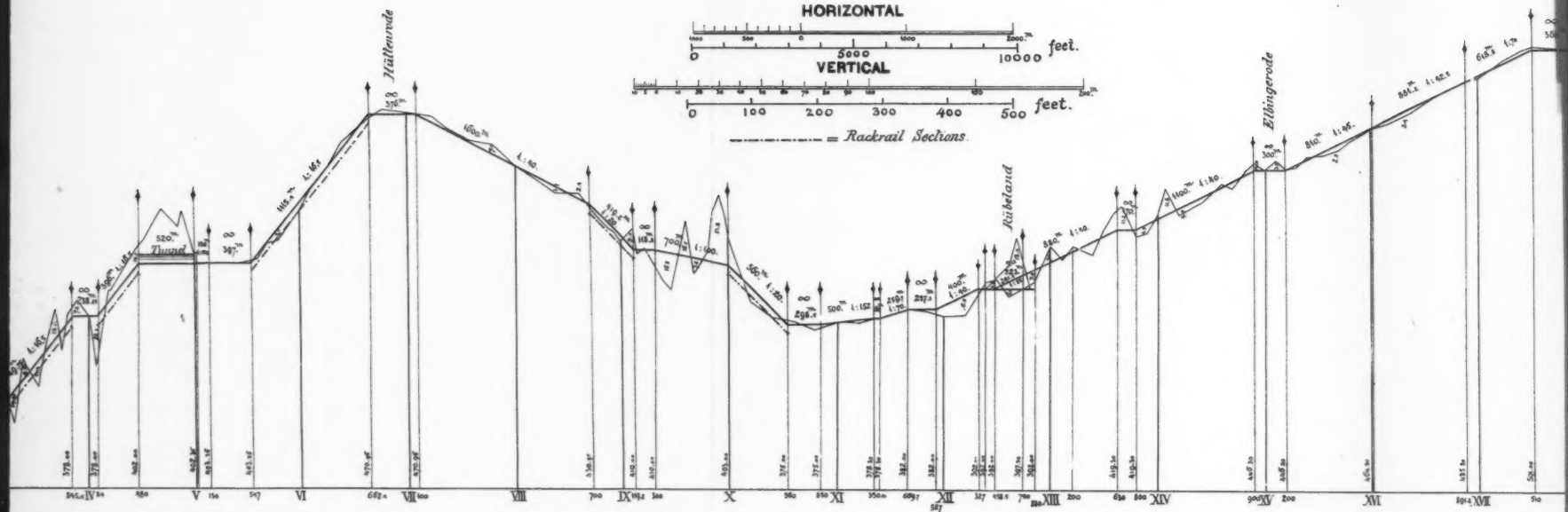
HORIZONTAL

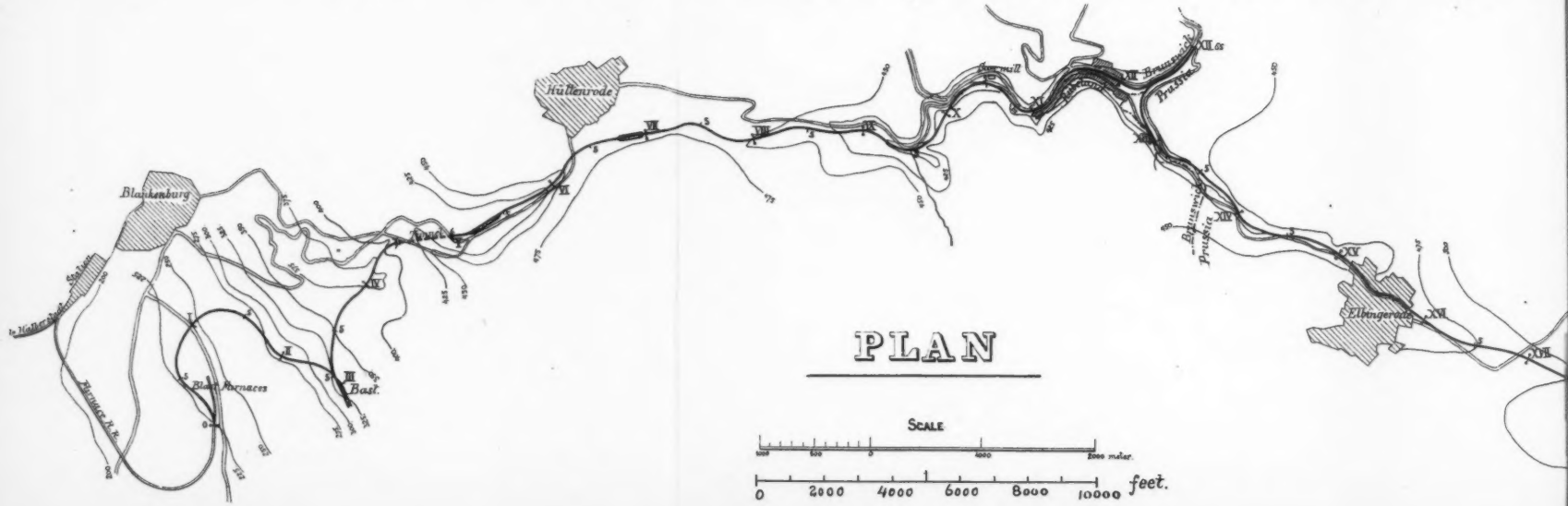
0 5000 10000 feet.

VERTICAL

0 100 200 300 400 500 feet.

= Rackrail Sections.





PLAN

SCALE

0 2000 4000 6000 8000 10000 feet.

PROFILE

PLATE VIII.
TRANS. AM. SOC. CIV. ENG'RS.
VOL XV. Nº 323
EVANS ON
ABT SYSTEM OF RAILWAY.

SCALES

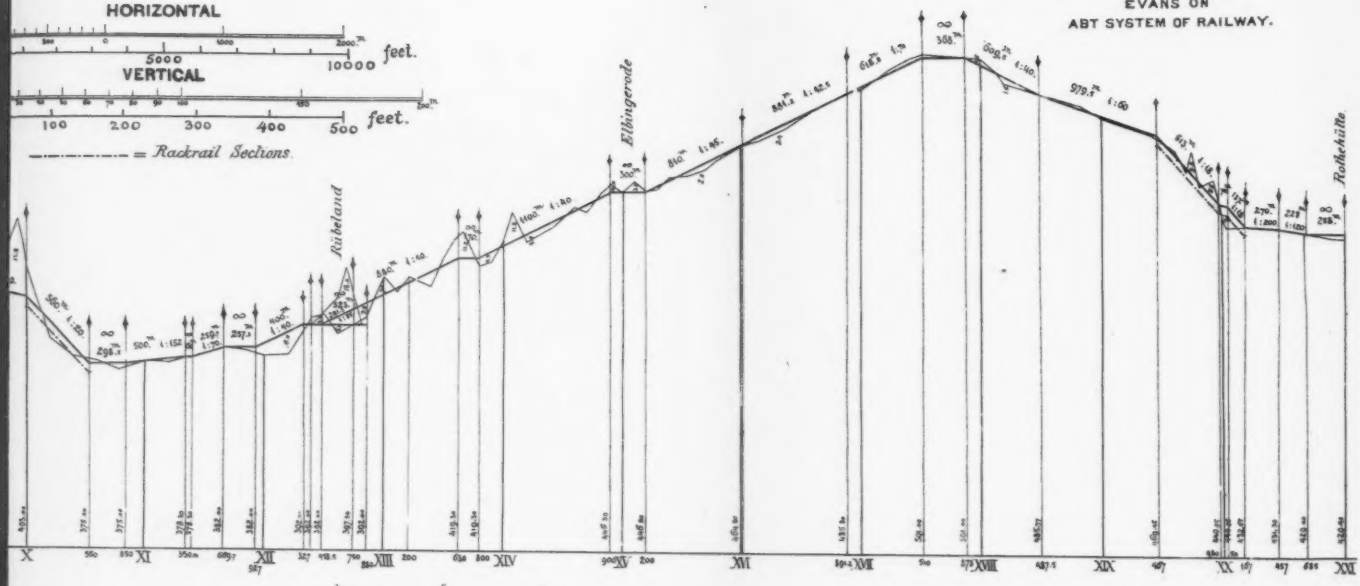
HORIZONTAL

5000 10000 feet.

VERTICAL

100 200 300 400 500 feet.

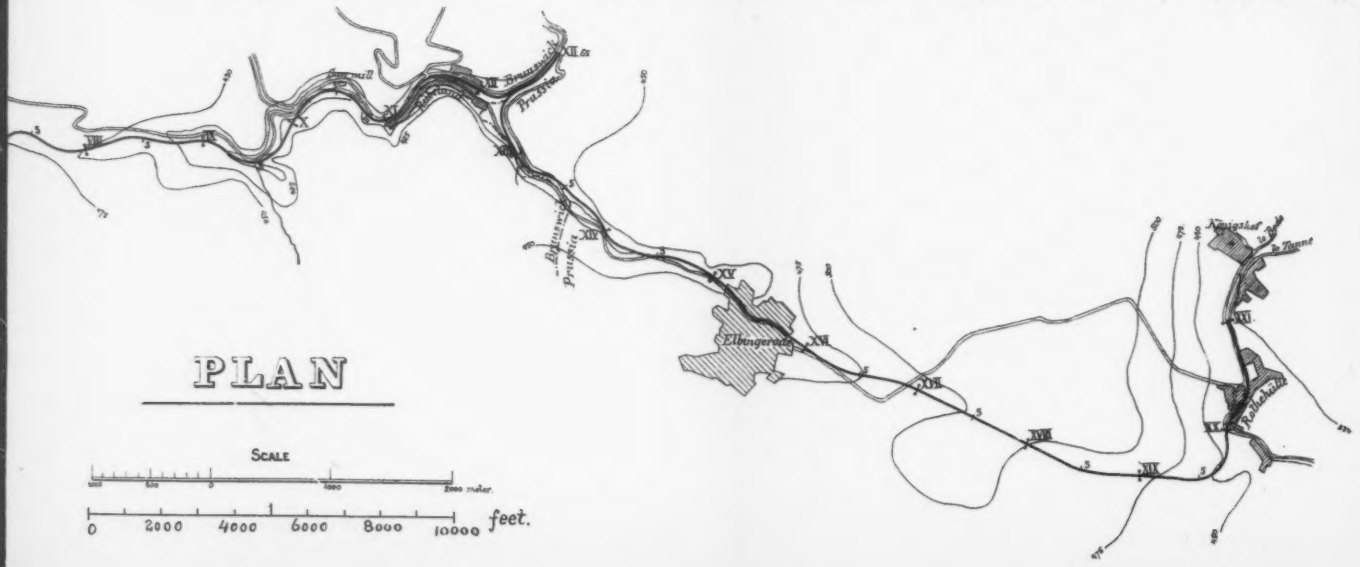
= Rackrail Sections.



PLAN

SCALE

0 2000 4000 6000 8000 10000 feet.



	Abt System.	Adhesion System.
Excavation (earth and rock),	330 000 cub. yds.	1 120 000 cub. yds.
Masonry (retaining walls, etc.)	5 640 "	33 900 "

The amounts are especially striking in the items of excavation and masonry, the former being about 3½ times and the latter about 6 times larger for the pure adhesion line than for the Abt system. Real estate runs in proportion to the length of the line.

Estimating at the rates there existing, and considering only those construction accounts which differ with the two systems, we find:

	Abt System.	Adhesion.
I.—Acquisition of real estate	$11 875	$16 500
II. Earthwork, rock, etc	113 825	308 668
Retaining walls	16 125	97 125
III.—Crossings	20 424	38 845
VI.—Bridges and culverts	2 704	5 414
X.—Superstructure, rails, ties, etc	97 504	97 500
XI.—Signals and telegraphs	405	562
	$262 862	$564 611

All other construction expenses, as for instance, for depot buildings, etc., are alike for both systems. The total saving, therefore, resulting from the application of the Abt system amounts to $301 749, or as the adhesion line would have had a length of $9\frac{32}{100}$ miles, we can say that in this case a saving of about $32 500 per mile of road has been obtained!

According to the stipulations of the contract, the locomotive must haul trains of 120 gross tons, exclusive of engine weight, over the entire road, and maintain a speed of 7½ miles on the 1 in 16.5 grades. The gross traction of the locomotive on this grade is 26 000 pounds, of which one-half is due to the adhesion drivers, and the other half is furnished by the pinion. A general characteristic description of the locomotives of this railway has already been given on page 162. The following are the principal dimensions:

Cylinder dimensions for adhesion	18 × 23½ in.
" " " pinions	12 × 23½ in.
Diameter of drivers	49 in.
Pitch line diameter of pinions	22½ in.
Pressure in boiler above atmosphere	150 pounds.
Heating surface	1 610 square feet.
Grate surface	19.4 "

Total weight in working order.....	54	gross tons.
Weight on adhesion drivers.....	42	"
Weight per driving axle	14	"

In designing these locomotives Mr. Abt had to adhere to the Prussian standards; if he could have followed the general locomotive practice the engines would have weighed from 49 to 50 gross tons.

The weight per driving axle may, perhaps, appear a little high, yet weights of 15 to 16 tons are of common occurrence on English lines, and that too with fast passenger engines. Considering the comparatively small velocities on such mountain lines, a weight of 14 tons, which is little influenced by the balancing parts is entirely justified.

Passenger and freight cars are equipped with a continuous train-brake; they are also provided with pinions for braking purposes.

On the 15th and 16th of last May, an experimental train of the regulation 120 gross tons, exclusive of engine weight, was run over the portion of the line which had been sufficiently finished for this purpose. This part contains a gradient of 1 in 16.5, and the change into this gradient takes place from a short horizontal section; on the gradient the line follows a curve. The locomotive maintained with ease the prescribed velocity on this gradient; then the train was stopped, backed, started again, in short it was manœuvred on this incline of 1 in 16.5 with the same facility as on a line of easy gradients. The train was also run with speeds up to 16 miles per hour on the rack-rail, and the working of the pinion in the rack was as noiseless as the working of the adhesion drivers on the ordinary rails. There were no blows, tremors or vibrations in any part, everything worked smoothly and evenly. The entering rack-rail operated to perfection, and the whole was a complete success.

Since then prominent engineers and engineering societies from various parts of the Continent have visited this novel railway, and they all unite in pronouncing it to be one of the most important and far-reaching additions to our modern railway service.

Höllenthal Railway.

A second railway to which Abt's system will be applied is being built by the government of Baden, South Germany. It commences at Freiburg and will extend for the present to Neustadt. Its length is 21¾ miles, 4 miles of which will be laid with the rack-rail; maximum gradient 290 feet

per mile. Gauge of line 4 feet 8½ inches. This road, generally known in Germany as the "Höllenthal Railway," so called from the "Höllenthal," a picturesque and romantic gorge of the Black Forest through which it runs, has been in contemplation for a great number of years. The difficulties of its location, with the adoption of Abt's system, have ceased to exist.

Both these lines, the Harz Railway and the Höllenthal Railway, will be members of the German Railroad Union, and as such, will be open to all continental rolling stock.

Railways in Bavaria.

In Lehesten, Bavaria, extensive slate quarries are being connected with the Bavarian Government Railways by a line upon Abt's system. The gauge is 4 feet 8½ inches, as the rolling stock of the government railroads is to be admitted. The line has a length of 2¾ miles, of which ¾ mile is to be laid with the rack-rail on gradients of $422\frac{4}{10}$ feet to the mile. The rack-rail is composed of only two elementary bars. The engine will have four adhesion drivers, two driving pinions and two truck wheels. The weight in complete working order will be 20 to 21 gross tons. The normal train weight consists of 50 gross tons, equivalent to three loaded cars. The line will be finished and in working order by November of this year.

At the same time a number of narrow-gauge tracks are being laid in the slate quarries themselves. Gauge, 27½ inches. Maximum gradients for rack-rail, 721 feet to the mile. It is on this network of tracks that the little locomotive of but 4½ gross tons will have to work.

Since writing the foregoing, a translation has been sent to the writer from a German paper, in which Mr. Schneider, Herzoglich Braunschweigischer Bahndirector (under whose official inspection the Harz Railway is being constructed), says:

"All throughout the Harz the name of 'Abt' is mentioned only with a deep feeling of veneration and gratitude, and for time to come many inhabitants of other mountain districts will do likewise."

Notes.

First.—Richard Trevithick, the engineer referred to in the first part of this paper, having made for himself a fine reputation in the matter of steam-pumps, dropped railways, and on being invited to Peru, went there. He succeeded in pumping the water from some rich mines that had been for a long time flooded. The delight of the owners and the enthusiasm of the Viceroy were so great, that a statue of solid silver was

ordered to be erected to him in the Plaza of Lima. Not having seen anything of the kind there, the writer asked what had become of the statue, and was amazed to find that not one of the "Hidalgos" of that once famous city had ever heard of the statue or of Trevithick either. It must be recollected in connection with this, that Trevithick was in Peru in the second decade of this century, not such a long time ago.

Second.—The Panama Railway has been referred to as the line on which George Escol Sellers' center-rail and horizontal driving-wheels were to be first used.

John L. Stephens, the projector and promotor of that line, counted on having to pass over a mountain range and encounter very severe gradients. No line had yet been run nor had accurate levels been taken. At the close of 1848 the writer loaned him one of his assistants, James Baldwin, to run a line and take levels. It was Baldwin who discovered the present location, and the summit of the Panama Railway is known as Baldwin's summit to this day. On this very route Count Ferdinand De Lesseps is now vainly attempting to build a sea-level canal.

Sellers' engines were built for and were the first used on that railway, but they ran only as ordinary adhesion engines. It may be interesting to know that Sellers prosecuted the railway company for not building heavy gradients and using his system, and he actually got a judgment in his favor for $14,000. This information was given to the writer by that Nestor of American engineering, Horatio Allen, Hon. M. Am. Soc. C. E., who was consulting engineer to the Panama Railway Company. It may also be very interesting to the Members of the Society to know that there is still alive and in the possession of great physical and mental ability—now when the world is covered with railways and has taken on itself a new phase of its existence—a man who has walked with George Stephenson over the Liverpool and Manchester Railway during its construction. This man is Horatio Allen. It was he who gave steam and life to the first locomotive ever moved on the American continent; and the writer, your humble servant, had the honor to give steam and life to the first locomotive ever moved south of the equator.

Third.—In April, 1873, Dr. W. Poole, F. R. S., read a paper on the Rigi Railway of Switzerland before the Institution of Civil Engineers of England. In that paper he gives the entire credit of that railway and its peculiar track and rolling stock to Mr. Riggenbach as the inventor. He was not to blame, as he had (the writer has also) seen the words "Système Riggenbach" on the locomotives of that line and elsewhere. Knowing that a most elaborate and detailed report, with numerous graphic pen sketches of every part of the Mount Washington Railway had been made by a Swiss engineer (referred to in the preceding paper), and had been sent to Mr. Riggenbach, the writer saw clearly that Mr. Riggenbach was stealing the thunder that properly belonged to Mr. S. Marsh. The Swiss engineer, Herr Otto Grüninger, kindly consented to give the writer a translation from his German press copy of that report. Another copy the writer had made and bound as a book, to send to the Institution of Civil Engineers in London, where it is on record as evidence of what Mr. Marsh first did with a rack-rail in our time.